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JOURNAL

SANITARY ENGINEERING

Proceedings of the American Society of Civil Engineers

DESIGN OF VENTURI FLUMES IN CIRCULAR CONDUITS

Edwin A. Wells[1] and Harold B. Gotaas,[2] M. ASCE
(Proc. Paper 928)

SYNOPSIS

The results of extensive experimental studies to determine the accuracy and design criteria for Venturi flumes of the type first recommended by Palmer and Bowlus are presented. The coefficient of discharge for the flumes and the hydraulic aspects of Venturi flow, such as energy criteria, channel slope, submergence, and velocity of approach, are discussed. The experimental studies and results show the influence of various flume dimensions such as throat length, side slope, base height, transitions, and point of depth measurement on the accuracy of the flume.

INTRODUCTION

During the past forty years, many Venturi flumes have been installed for the measurement of flow of sewage and irrigation waters in open channels. The method design for most of these flumes can be traced to one of three sources: (a) English flumes, rectangular in shape which were based upon early work in India (1908-1914) and the writings of F.V.A.E. Engal(1) in England (1933); (b) The Parshall Flume originated by Cone(2)(3) (1915), modified and tested extensively by Parshall(4)(5)(6) (1917-1927); and (c) flumes of the type first suggested and developed by Harold K. Palmer and Fred D. Bowlus(7) (1936).

This paper reports the results of laboratory experiments which were conducted to provide data on the accuracy of the Palmer-Bowlus flume and on the importance of various design criteria. This type of Venturi flume is characterized by a throat of uniform cross section and a length approximately equal to one diameter of the pipe in which it is to be installed. Its shape, as suggested by Palmer and Bowlus, has been found to make it inexpensive to

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construct, simple to operate, easy to install and maintain, accurate in its measurement of flow, and low in energy loss. Although it is not as widely known as the Parshall flume, many Palmer-Bowlus flumes have been installed in the western United States during the past twenty years.

Seven flumes conforming to the Palmer-Bowlus flume shape were built and tested to determine the accuracy of the flume under normal operating conditions. One of these flumes was tested extensively under varying conditions of channel slope. A second series of flumes was built with interchangeable parts to study the importance of flume dimensions such as throat length, base height, and entrance transition. Each of the design variable is discussed separately, in the paper.

Derivation of Laws

Rating Curve

Using the sketch of Figure 1 as a basis for analysis of the flow in the Venturi flume, the Bernoulli equation for the energy at the two sections indicated may be written as follows:

$$E_1 = d_1 + \frac{V_1^2}{2g}$$

$$E_2 = d_2 + \frac{V_2^2}{2g} + t$$

Equating these and solving for the depth of flow at Section 1:

$$d_1 = d_2 + t + \frac{V_2^2}{2g} - \frac{V_1^2}{2g} \quad \cdots\cdots\cdots (1)$$

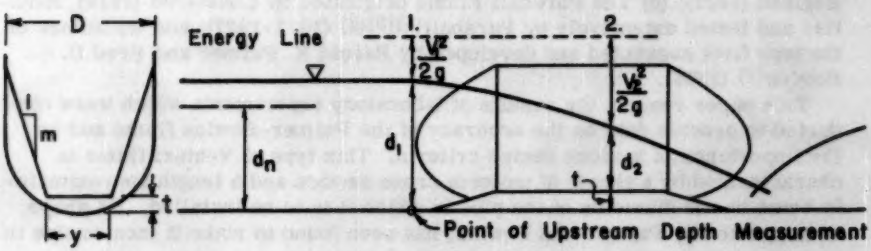


FIGURE 1

Since the flume should operate under the free discharge condition, the flow must pass through critical depth in the throat where:

$$d_2 = d_c \quad \text{and}$$

$$\frac{V_2^2}{2g} = \frac{V_c^2}{2g} = \frac{A_c}{2B_c}$$

Equation (1) may be rewritten

$$d_1 = d_c + t + \frac{A_c}{2B_c} - \frac{V_1^2}{2g} \quad \cdots\cdots (2)$$

For a given flow rate, Q, the variable quantities $\frac{V_1^2}{2g}$, d_c, and $\frac{A_c}{2B_c}$ have fixed values hence equation (2) may be written

$$Q = f(d_1) \quad \cdots\cdots (3)$$

Several methods have been suggested for the solution of equation (3). The method proposed by Arredi[8] and clarified by H. F. Ludwig[9] and the Los Angeles County Sanitation Districts was used in this investigation, since it is considered to be more easily adapted to irregular shaped sections than other procedures. Another convenient method has been proposed by J. H. Ludwig and R. C. Ludwig.[10] An Arredi diagram and calculations for rating a flume are presented in Appendix 1.

A calculated "rating" curve of $Q = f(d_1)$ for a given Venturi flume installation will hereafter be referred to as the theoretical curve of that flume. It should be noted that the theoretical curve is based on a "U"-shaped channel rather than circular channel. If the sides of the throat are made up of planes which extend unbroken to the top of the channel, equation (3) has the logarithmic form $Q = kd_1$.

In deriving equations (2) and (3) for rating the flume, several assumptions have been made. The most important of these are: (a) that there are no energy losses between Sections 1 and 2, and (b) that the streamlines are parallel, and are perpendicular to both Section 1 and Section 2. Actual flow conditions vary slightly from these assumptions since side-wall friction and other energy losses influence the flow through the flume. The character of the flow at Section 2 is at variance with the original assumption unless correction factors are used, since the Bernoulli equation is valid only where there is no curvature of the streamlines. It is apparent that the use of the "average velocity" and the "velocity head" when the actual velocity distribution has not been considered is a simplification of the conditions that exist. The drawdown through Section 1, though small, also affects the upstream depth, d_1. The magnitude of the effect of these assumptions upon the accurate measurement of the quantity of water passing through the flume is given quantitatively by the discharge coefficient which as shown later does not vary more than 3% from the theoretical value for most standard flumes. For this investigation, a flume was said to be performing satisfactorily when the coefficient of discharge did not exceed a range of from 0.95 to 1.05.

Energy Criteria

The rate of flow, Q, becomes a function of the upstream depth, d_1, when the specific energy, E_1, of the flow in the pipe is equal to the specific energy, E_2, of the same flow at critical depth in the throat of the flume. On the basis of this criterion, the theoretical curve of flow vs depth for the flume may be computed without regard to the conditions of actual usage. However, to install a flume in a given channel so that it will operate properly, additional energy criteria must be satisfied.

The principle of the Venturi flume as well as the energy criteria essential for its proper design and operation may be described by the use of a specific energy diagram. Figure 2 shows the specific energy curves for a given rate of flow in the U-shaped channel and in the throat of a Venturi flume. Figure 2 also shows the possible conditions of normal flow in the channel prior to the installation of a flume, separated into four zones based on depth of normal flow. In all cases the dimensions of the channel and the flume, the rate of flow, and therefore the theoretical upstream depth, d_1, are held constant.

Zone 1. Tranquil Channel Flow - Submerged Discharge: If the normal depth, d_n, of flow in the channel without the flume is greater than, d_1, the constriction produced by the flume will merely create a depression in the water surface which is equal to the increase in velocity head. Thus the flow would not pass through critical depth in the flume, a condition which is referred to as submerged discharge.

Zone 2. Tranquil Channel Flow - Free Discharge: In the usual case, the normal depth of flow, d_n, in the channel is less than the upstream depth, d_1, required for operation of the flume and is greater than the critical depth, d_{cc}, in the channel. Installation of the flume in the channel will cause water to back up until it reaches depth d_1. From this depth, the water flows into the flume, passing through critical depth within the throat of the flume. To assure that the flume is not submerged, the normal depth of flow in the channel should be somewhat less than the upstream depth. (Approx. 0.85 d_1 or less.)

Zone 3. Rapid Channel Flow - Free Discharge: If the normal depth of flow in the channel is less than the critical depth, d_{cc}, placing the flume in the channel will cause the water to jump to its conjugate depth with a subsequent loss of energy. The minimum normal depth, $d_{n\ min.}$, shown on the curve represents the minimum depth of normal flow for which the flume will continue to function properly. In this case the difference in specific energies $E_3 - E_{ct}$ represents the loss of energy in the jump as the water surface passes from $d_{n\ min.}$ to d_1. This would be typical of the case where a jump occurred immediately upstream from the flume. For other instances where d_n is less than d_{cc} and greater than $d_{n\ min.}$, the hydraulic jump would occur further upstream from the flume, and the water surface would back up at the flume to the depth d_1.

Zone 4. Rapid Channel Flow - Unstable Discharge: Where the normal depth of flow, d_n, is less than $d_{n\ min.}$ and the energy of the flow is correspondingly greater than E_3, say E_4, energy $E_4 - E_3$ is dissipated by turbulence in the upstream channel after the jump has occurred. If the energy of the flow exceeds E_3, the conditions of discharge are

SPECIFIC ENERGY CURVES FOR "U" SHAPED CHANNEL AND VENTURI FLUME THROAT

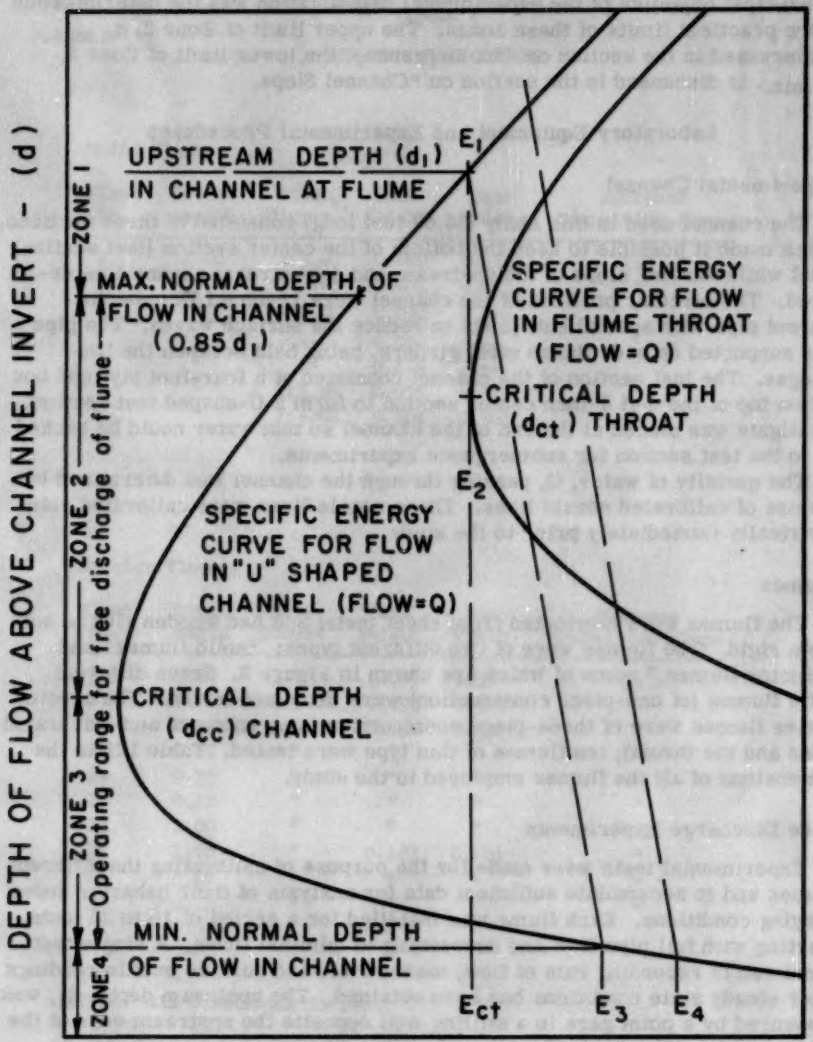

DEPTH OF FLOW ABOVE CHANNEL INVERT - (d)
SPECIFIC ENERGY - (E)
ZONE 1
ZONE 2
ZONE 3
ZONE 4
Operating range for free discharge of flume
UPSTREAM DEPTH (d_1) IN CHANNEL AT FLUME
MAX. NORMAL DEPTH OF FLOW IN CHANNEL (0.85 d_1)
CRITICAL DEPTH (d_{cc}) CHANNEL
MIN. NORMAL DEPTH OF FLOW IN CHANNEL
SPECIFIC ENERGY CURVE FOR FLOW IN "U" SHAPED CHANNEL (FLOW=Q)
SPECIFIC ENERGY CURVE FOR FLOW IN FLUME THROAT (FLOW=Q)
CRITICAL DEPTH (d_{ct}) THROAT
E_1
E_2
E_{ct}
E_3
E_4

FIG. 2

unstable and do not conform to the expression $Q = f(d_1)$. Conditions may be such that no pond forms upstream and there is shooting flow through the upstream transition of the flume.

Satisfactory operation of the flume can be obtained only in Zones 2 and 3. A principal objective of the experimental investigation was the determination of the practical limits of these zones. The upper limit of Zone 2, $d_{n\ max.}$, is discussed in the section on "Submergence," the lower limit of Zone 3, $d_{n\ min.}$, is discussed in the section on "Channel Slope.

Laboratory Equipment and Experimental Procedures

Experimental Channel

The channel used in this study (36.67 feet long) consisted of three sections, which made it possible to keep the bottom of the center section (test section) level while varying slope of the upstream and downstream channels as desired. The circular portions of the channel were 12-inch I.D. asbestos-cement pipe with special butt joints to reduce the surface waves. The pipe was supported on wide flange steel girders, being held between the two flanges. The test section of the channel consisted of a four-foot plywood box set on top of the 4-ft 6-inch center section to form a U-shaped test section. A tailgate was placed at the end of the channel so that water could be backed up to the test section for submergence experiments.

The quantity of water, Q, passing through the channel was determined by the use of calibrated nozzle lines. These nozzle lines were calibrated volumetrically immediately prior to the study.

Flumes

The flumes were fabricated from sheet metal and had wooden ribs to hold them rigid. The flumes were of two different types: "solid flumes" and "erector flumes," some of which are shown in Figure 3. Seven different solid flumes (of one-piece construction) were made and tested. The erector series flumes were of three-piece construction (i.e., entrance and exit transitions and the throat); ten flumes of this type were tested. Table I lists the dimensions of all the flumes employed in the study.

Free Discharge Experiments

Experimental tests were made for the purpose of calibrating the different flumes and to accumulate sufficient data for analysis of their behavior under varying conditions. Each flume was installed for a series of 15 to 25 tests, starting with full pipe flow and decreasing to minimal flows. A single test consisted of recording rate of flow, manometer, and surface profile readings after steady state conditions had been obtained. The upstream depth, d_1, was measured by a point gage in a stilling well opposite the upstream edge of the entrance transition to the flume.

The values of Q and d_1 plotted to show the actual flow as a function of measured depth is the "calibration" or "actual test" curve for a flume.

All flumes were checked by two or more series of tests. In many of the test curves shown, the flume was removed after the first test and an additional test series run under similar conditions at a later date. In no case did a flume fail to reproduce the calibration curve obtained from earlier runs.

TABLE I

DIMENSIONS OF FLUMES

Solid Flumes:

Flume No.	Throat Length	Side Slopes	Base Height	Base Width	Entrance Transition Slope
1.	1.00	1:3.07	0.089	0.333	1:3
3.	"	1:2.05	0.083	0.333	"
4.	"	1:1.00	0.096	0.321	"
5.	"	--	0.089	--	"
6.	"	--	0.179	--	"
7.	"	1:3.04	0.035	0.333	"
8.	"	1:3.02	0.184	0.343	"

Erector Flumes:

Flume No.	Throat Length	Side Slopes	Base Height	Base Width	Entrance Transition Slope
1a	2.00	1:3	0.095	0.354	1:3
1b	1.50	1:3.08	0.093	0.340	"
1c	1.00	1:3.20	0.090	0.337	"
1d	0.67	"	"	"	"
1e	0.33	"	"	"	"
1f	0.12	"	"	"	"
1g	1.00	"	"	"	1:2
2c	1.00	"	0.187	0.401	"
2d	0.67	"	"	"	"
2f	0.12	"	"	"	"

NOTE: All Dimensions in feet.

TABLE II

TEST CONDITIONS FOR VARIOUS FLUMES

Type of Test	Flume No.	Slope*	Number of Test Runs
Solid Flume Test	1	0.1%; 0.5%	97
	3	0.1%; 0.5%	112
	4	0.1%; 0.5%	89
	5	0.1%; 0.5%	97
	6	0.1%	42
	7	0.1%	58
	8	0.1%	53
Erector Model Test	1a	0.1%	62
	1b	0.1%	33
	1c	0.5%	128
	"	2.0%	107
	"	3.0%	67
	"	4.0%	41
	"	4.5%	17
	"	5.0%	10
	1d	0.5%	39
	1e	0.5%	21
	1f	0.5%	36
	2c	0.5%	40
	2d	0.5%	41
	2f	0.1%	22
Submergence Test	1	0.1%	15
	3	0.1%	18
	6	0.1%	19
	7	0.1%	14
	8	0.1%	12
	1c	0.5%	35
Point of Depth measurement			
12-inches downstream from flume entrance	1	0.1%	23
24- " " "	1	0.1%	21
No Terminal Transition	1c	0.5%	17
1:2 Entrance Transition	1g	0.1%	22

*Indicated Channel Slope, not energy line slope.

Table II presents a summary of the conditions under which the various flumes were tested.

Submergence Tests

The following procedure was followed in conducting the submergence experiments.

A standard test run was made with the flume in a normal position. The point gage reading for the depth d_1 was carefully checked to find the mean reading. The tip of the point gage was then raised 0.005 foot and checked to see that normal surface undulations in the stilling well did not touch it. The tail gate at the end of the channel (15 ft. from the flume) was raised slowly until the submergence of the flume caused the water in and above the flume to rise and touch the point gage at d_1. The depth of water below the flume was then determined with a second point gage and the submergence ratio calculated. The channel was level from d_1 to the tail gate. The distance between the point gages was approximately ten feet.

Experimental Results and Analysis

Free Discharge Experiments

Nine different standard flume shapes were tested throughout a wide range of flows and energy conditions. A standard flume is defined as one having a throat length equal to the diameter of the pipe. The investigation of the behavior of these flumes under normal, free discharge conditions was one of the principal objectives of the study. Table III shows the flow range within which measurements were made for each flume. The measured upstream depth, d_1, and rate of flow, Q, which determine the actual test curve is compared with the theoretical discharge curve for each of these flumes in Figures 4, 5, 6, 7, 8 and 9. The correlation between the actual and theoretical curves is discussed under Coefficient of Discharge.

These flumes were tested under the most desirable operating conditions

TABLE III

FLOW RANGES FOR STANDARD FLUMES

Flume No.	Minimum Flow (cfs)	Maximum Flow (cfs)	Ratio Max. to Min.
8	0.18	1.16	6.5
2c	0.18	1.25	7.0
1c	0.18	1.49	8.3
1	0.18	1.55	8.8
7	0.18	1.78	9.9
3	0.17	2.10	11.7
4	0.17	2.70	15.0
6	0.18	2.70	15.0
5	0.18	2.80	15.5

NOTE: Maximum Flow read from "Actual Test Curve" at 0.95 feet depth.

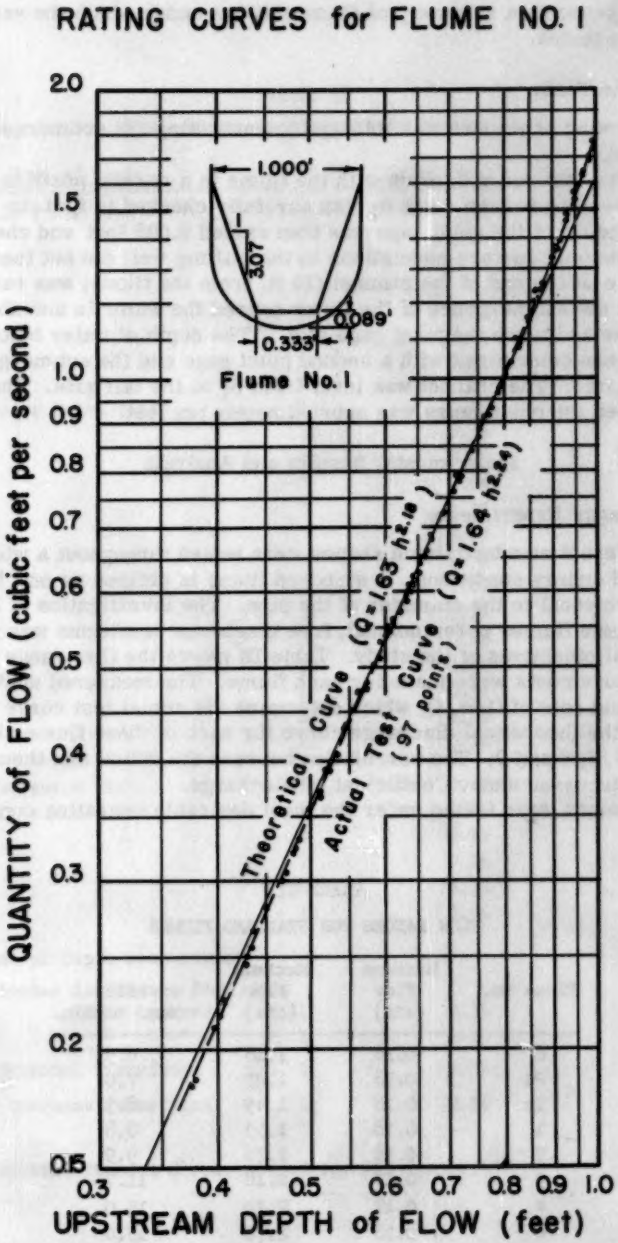

RATING CURVES for FLUME NO. 1
1.000'
1
3.07
0.089'
0.333'
Flume No. 1
Theoretical Curve (Q=1.63 h2.48)
Actual Test Curve (97 points)
(Q=1.64 h2.24)
QUANTITY of FLOW (cubic feet per second)
2.0
1.5
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.15
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
UPSTREAM DEPTH of FLOW (feet)

Figure 4

RATING CURVES for FLUME NO. I-c

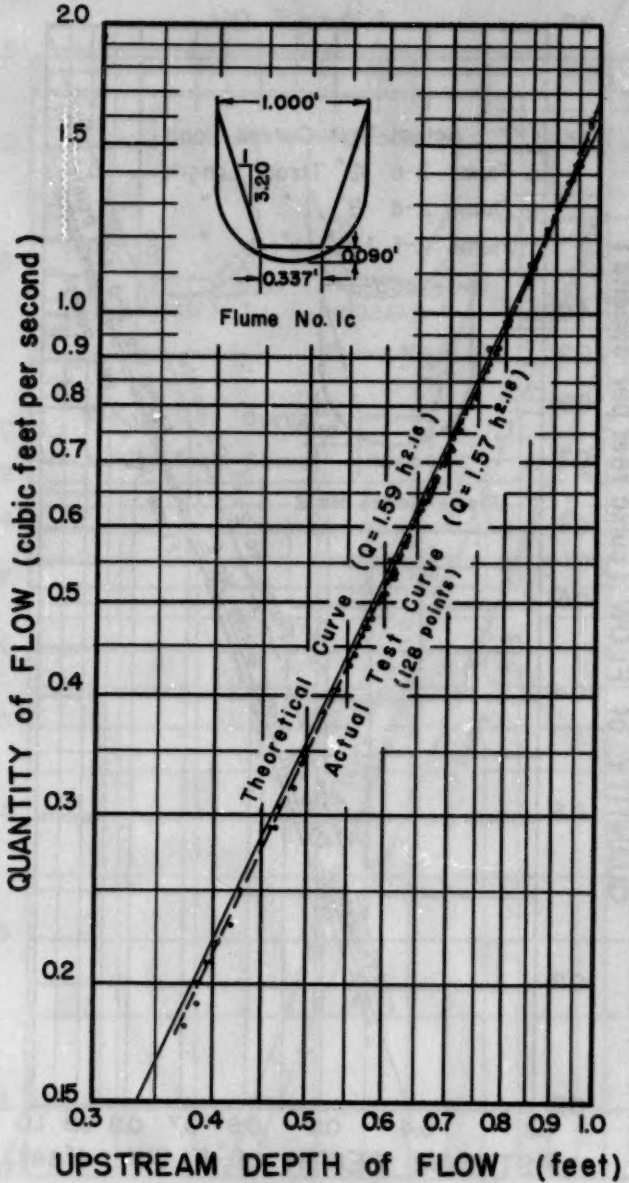


Figure 5

ERECTOR SERIES FLUMES NO. 2-c, d, f.

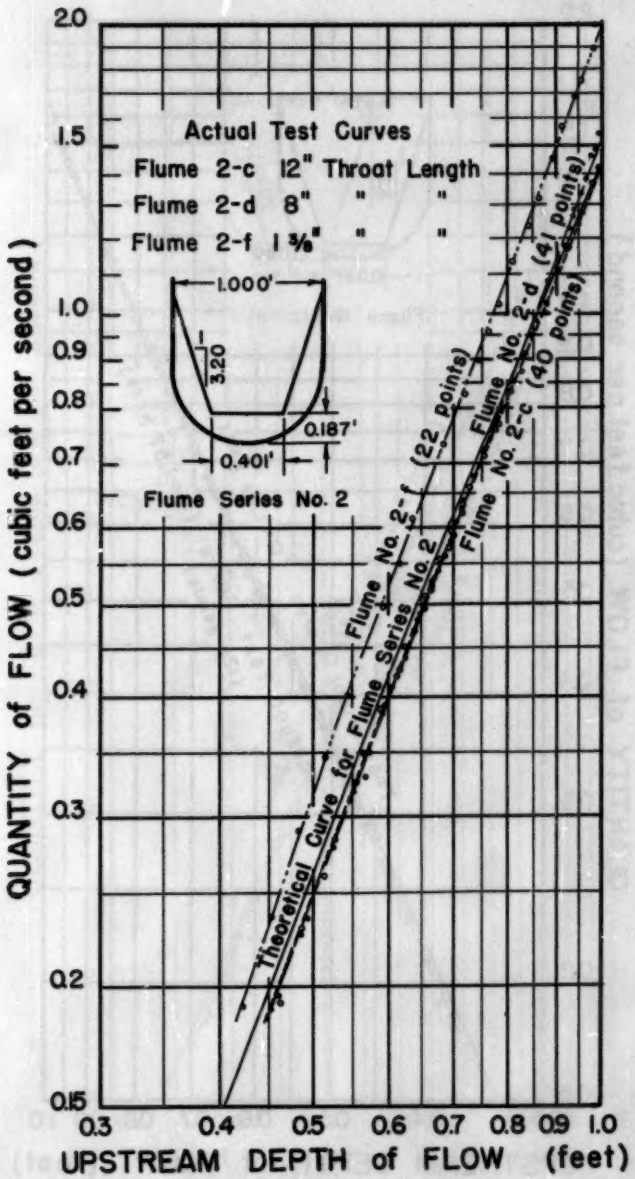


Figure 6

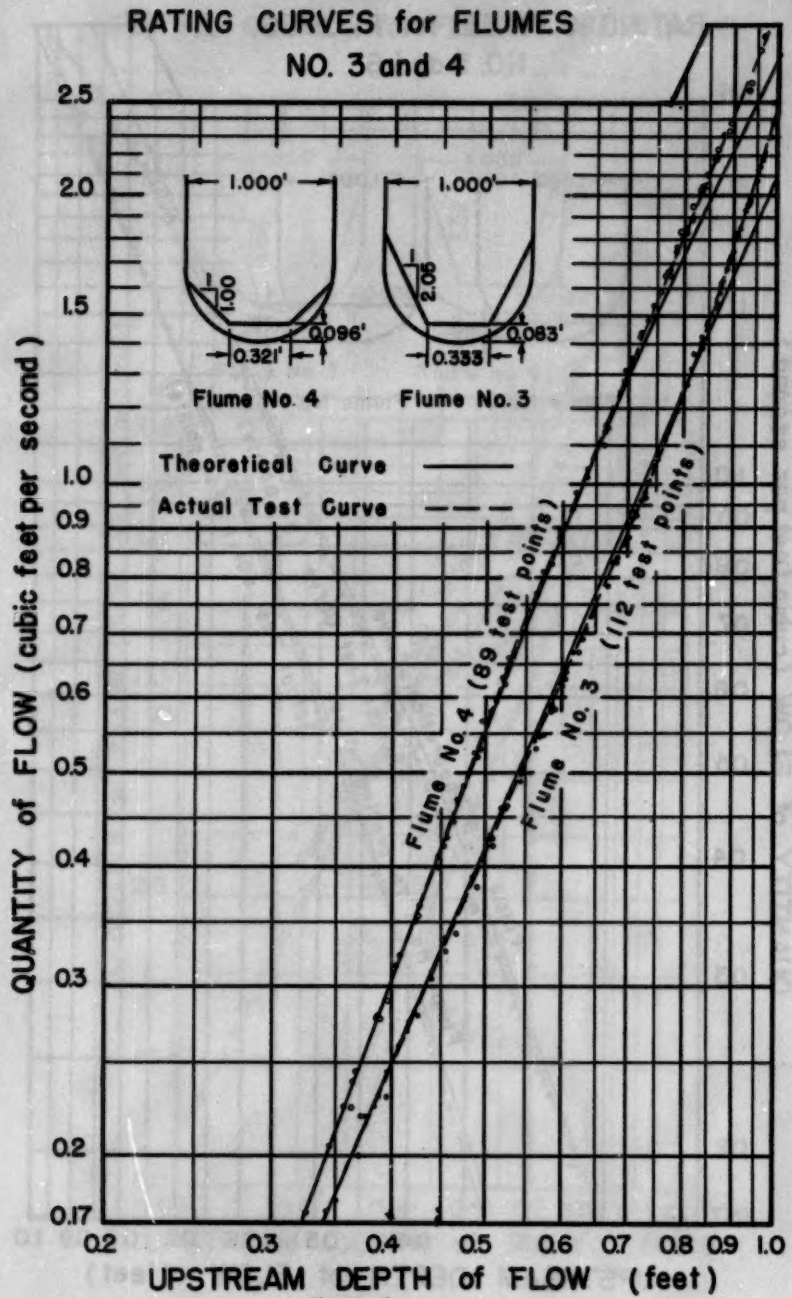

RATING CURVES for FLUMES
NO. 3 and 4
QUANTITY of FLOW (cubic feet per second)
UPSTREAM DEPTH of FLOW (feet)
2.5
2.0
1.5
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.17
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.000'
1.00
0.096'
0.321'
Flume No. 4
1.000'
2.00
0.083'
0.333'
Flume No. 3
Theoretical Curve
Actual Test Curve
Flume No. 4 (89 test points)
Flume No. 3 (112 test points)

Figure 7

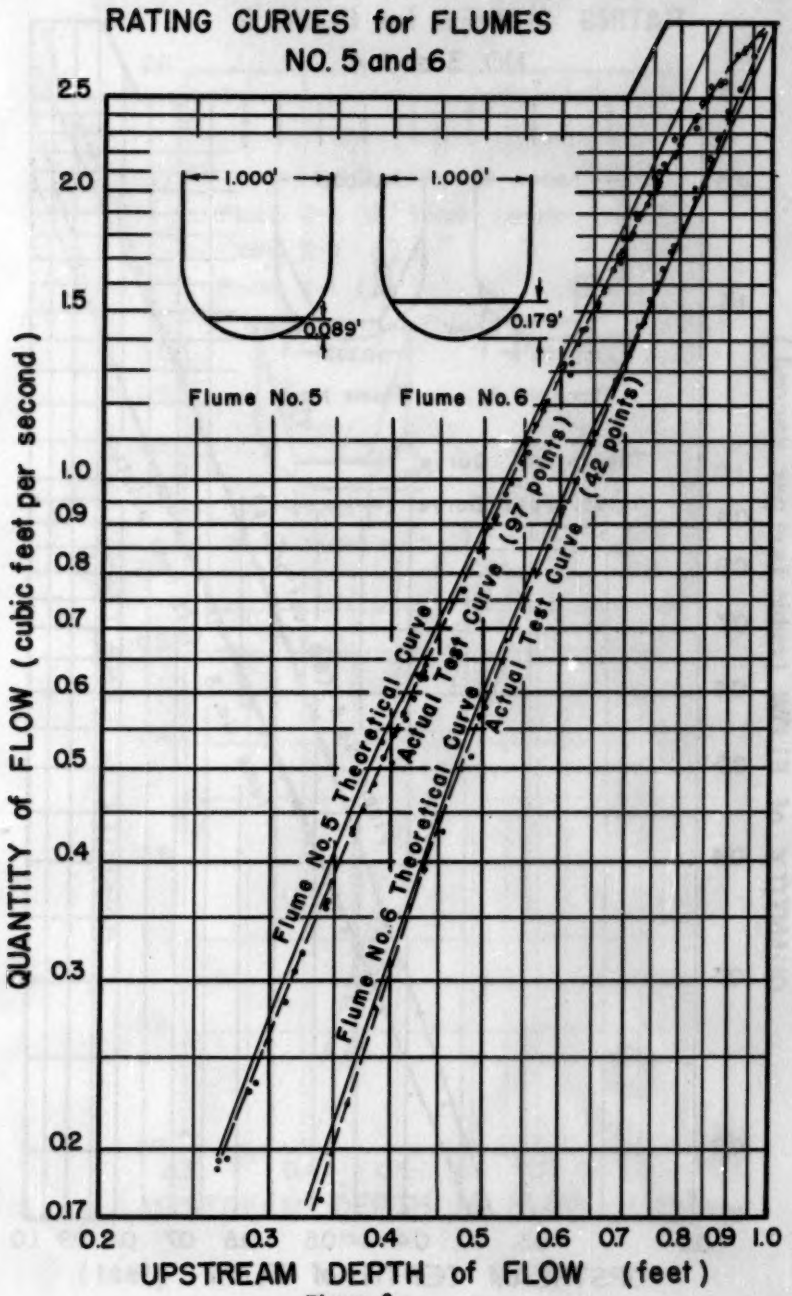

RATING CURVES for FLUMES
NO. 5 and 6
1.000'
1.000'
0.089'
0.179'
Flume No. 5
Flume No. 6
Flume No.5 Theoretical Curve
Actual Test Curve (97 points)
Flume No.6 Theoretical Curve
Actual Test Curve (42 points)
QUANTITY of FLOW (cubic feet per second)
2.5
2.0
1.5
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.17
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
UPSTREAM DEPTH of FLOW (feet)

Figure 6

RATING CURVES for FLUMES NO. 7 and 8

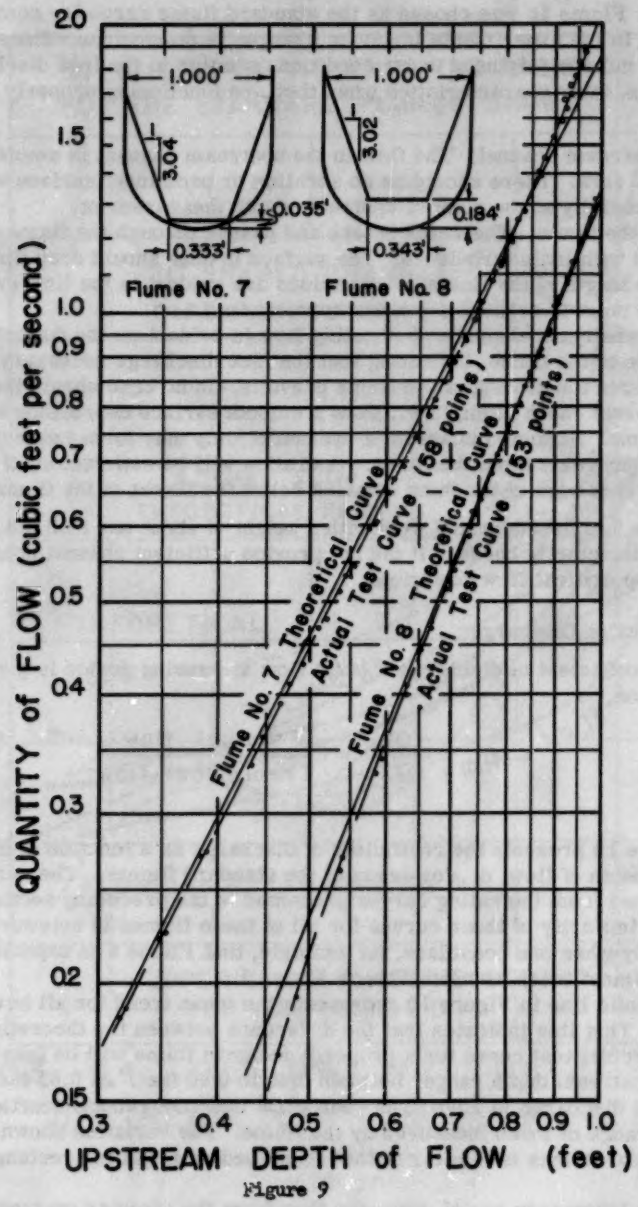


Figure 9

(an upstream energy line slope of less than 0.5%), and the data shown in Figures 4, 5, 6, 7, 8 and 9 were used as a basis of comparison of further tests on different aspects of flume design and operation. The actual test curve for Flume 1c was chosen as the standard flume curve for comparison purposes in the experiments involving changes in dimensions. Since flumes would be most widely used under conditions similar to the free discharge experiments, their characteristics when they are functioning properly are as follows:

1) Upstream channel: The flow in the upstream channel is smooth, tranquil flow. There should be no aeration or prominent surface waves, especially at the point of upstream depth measurement.
2) At the flume: The water enters and passes through the flume smoothly and with little turbulence. The surface profile should drop throughout the length of the flume. Streamlines are evident in the flow even after the point of critical depth has been reached.
3) Downstream channel: A shooting flow is evident on the downstream side of the flume, indicating that the free discharge necessary for proper functioning of the flume prevails. In no case should the flow merely "neck down," i.e., show a smooth surface depression at the flume. In some instances, a hydraulic jump may form immediately downstream from the flume. Operation will be satisfactory if the upstream edge of the jump remains below the throat of the flume.

Flume 5, a broadcrested weir with a height of about one inch did not perform satisfactorily because it did not provide sufficient channel construction to develop critical flow conditions.

Coefficient of Discharge

The coefficient of discharge C_d of a flow measuring device is given by the expression,

$$C_d = \frac{Q_a}{Q_t} = \frac{\text{Actual flow}}{\text{Theoretical flow}}$$

Figure 10 presents the coefficient of discharge as a function of the upstream depth of flow, d_1, for seven of the standard flumes. The curves were determined from the rating curves presented in the preceding section.

The similarity of these curves for all of these flumes is noteworthy especially when one considers, for example, that Flume 6 is capable of measuring almost twice as much flow as Flume 1.

The solid line in Figure 10 represents the mean trend for all seven flume shapes. This line indicates that the difference between the theoretical curve and the actual test curve for a properly designed flume will be less than 3% for an upstream depth ranges between 0.40 to 0.90 feet. At 0.82 feet the coefficient of discharge is 1.00. The coefficient does not remain constant throughout the range of flows measured by the flume. The variation shown here for trapezoidal flumes is similar to that described by Engal for rectangular flumes.

In the laboratory model, when the flow from the circular upstream channel entered the U-shaped test channel, flow conditions were not uniform in the top

STANDARD FLUME DISCHARGE COEFFICIENT AS A FUNCTION OF UPSTREAM DEPTH

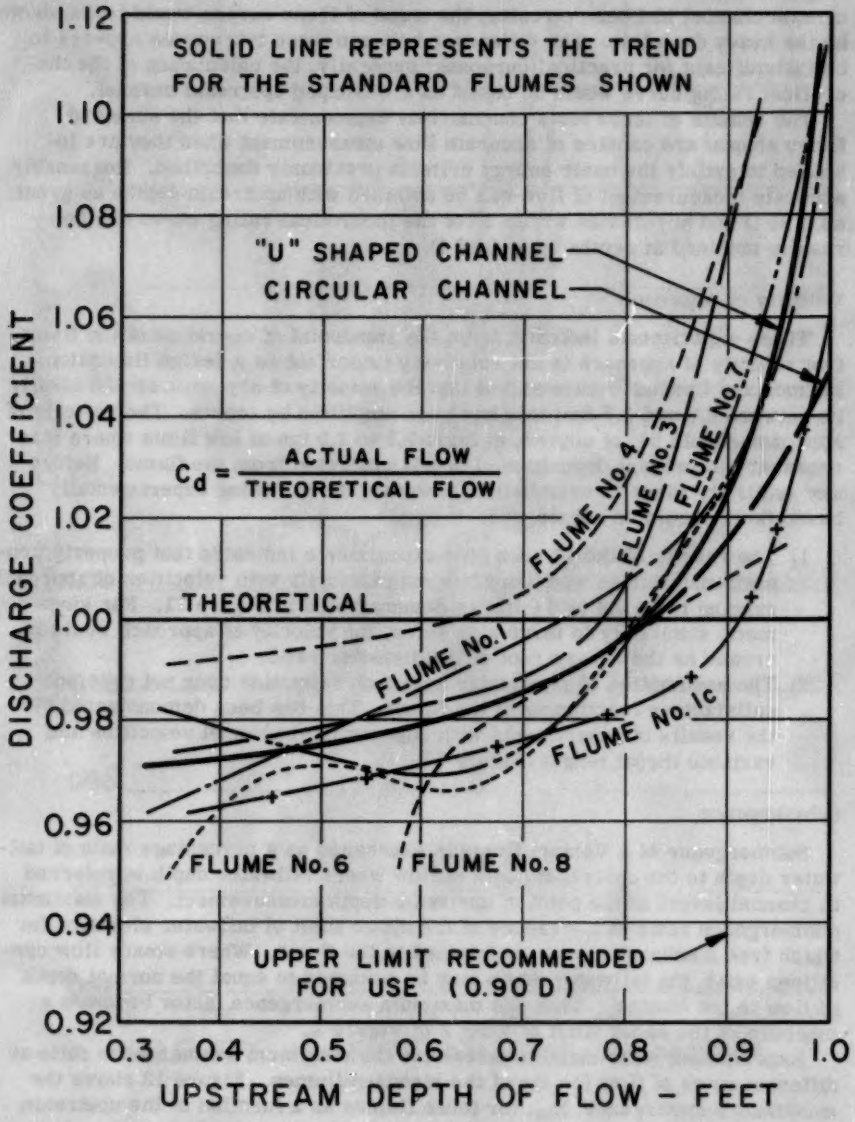


FIG. 10

surface of the test channel at upstream depths greater than 0.5 diameter. The curves shown in Figure 10 are based on a comparison of the actual test curve of the flume with a calculated rating curve which was, in turn, based on the assumption that the upstream channel was U-shaped and of sufficient length for uniform flow conditions to pertain. If it were assumed that the upstream channel had been circular, the trend of these curves would be as shown by the heavy dash line. The difference between these two curves appears to be insignificant for practical purposes; generally, the calculation of the theoretical rating curve would be based on a U-shaped upstream channel.

The results of these tests conclusively demonstrate that the standard flume shapes are capable of accurate flow measurement when they are installed to satisfy the basic energy criteria previously described. Reasonably accurate measurement of flow can be obtained with upstream depths as great as 0.95 D and accuracies within 3% of the theoretical rating curve can be readily obtained at depths up to 0.90 D.

Velocity of Approach

These experiments indicate, from the standpoint of operation of the flume, that velocity of approach is not relatively important as a design limitation. Palmer and Bowlus[7] have stated that the velocity of approach should ideally be between 2.0 and 2.5 fps; this has been amplified by others. The velocity of approach should be, of course, at least 1.5 to 2.0 fps at low flows where it is necessary to prevent deposition of solids upstream from the flume. Before any arbitrary range is established, however, the following experimentally based facts should be considered:

1) The results of the 12-inch pipe experiments indicated that properly proportioned flumes would operate satisfactorily with velocities of approach ranging from 0.8 to 3.0 fps as demonstrated in Figure 11. For kinematic similarity in large pipe sizes, the velocity of approach would increase as the square root of the diameter ratio.
2) The assumption of reasonable approach velocities does not preclude satisfactory functioning of the flume. This has been demonstrated by the results of experiments with supercritical channel velocities and variable throat length (Figure 17).

Submergence

Submergence of a Venturi flume is expressed as a percentage ratio of tailwater depth to the upstream depth of flow where tailwater depth is referred to channel invert at the point of upstream depth measurement. The maximum submergence ratio is a measure of the upper limit of tailwater elevation for which free discharge can be maintained in the flume. Where steady flow conditions exist, the tailwater depth may be assumed to equal the normal depth of flow in the channel. Thus the maximum submergence factor becomes a measure of the upper limit of Zone 2 of Figure 2.

Experiments were made to determine the maximum submergence ratio at different rates of flow for six of the standard flumes. Figure 12 shows the maximum submergence, S_m, for these flumes as a function of the upstream depth of flow. The results indicate that this is approximately a linear function through the range of depths shown (0.4 ft. to 0.95 ft.). In Figure 12, it can be seen that at a depth of 0.90 ft. the maximum submergence ratio is about 85% for the flumes tested.

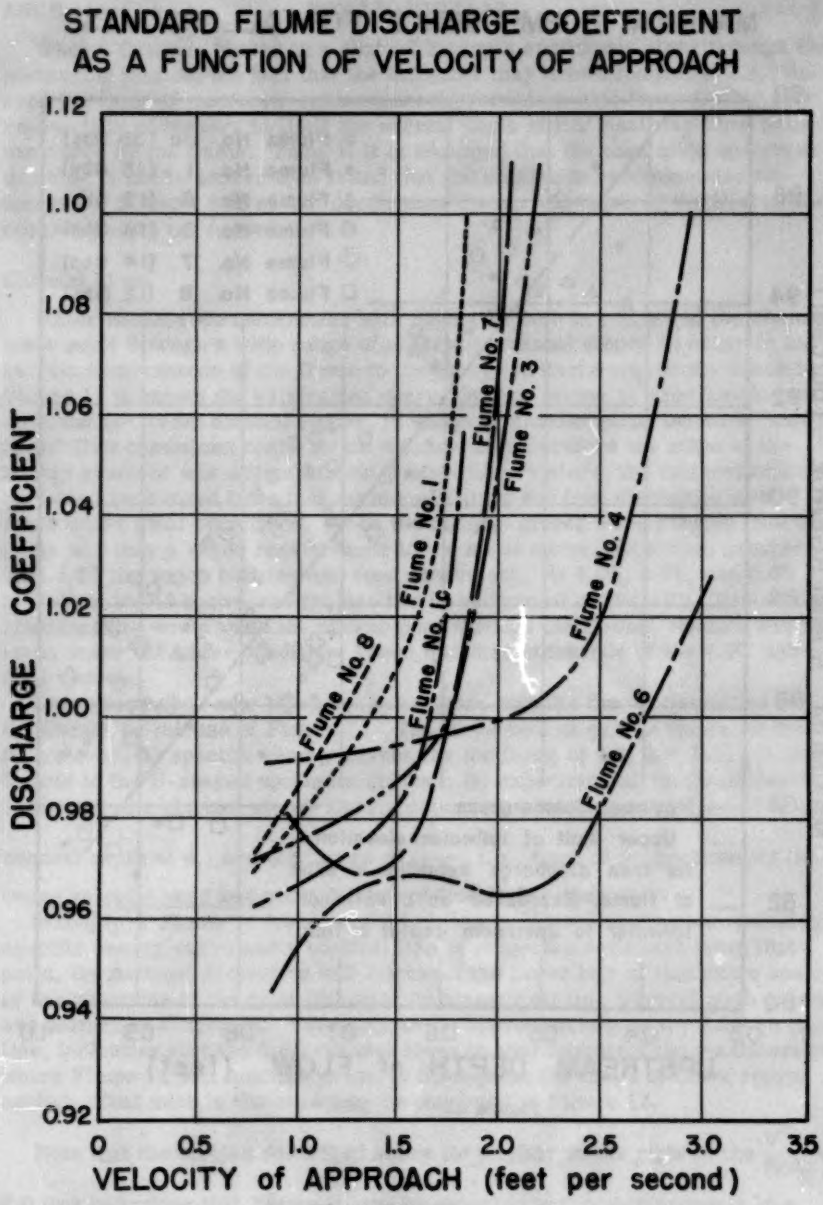

STANDARD FLUME DISCHARGE COEFFICIENT
AS A FUNCTION OF VELOCITY OF APPROACH
DISCHARGE COEFFICIENT
1.12
1.10
1.08
1.06
1.04
1.02
1.00
0.98
0.96
0.94
0.92
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
VELOCITY of APPROACH (feet per second)
Flume No. 1
Flume No. 7
Flume No. 3
Flume No. 4
Flume No. 8
Flume No. 5
Flume No. 6

Figure 11

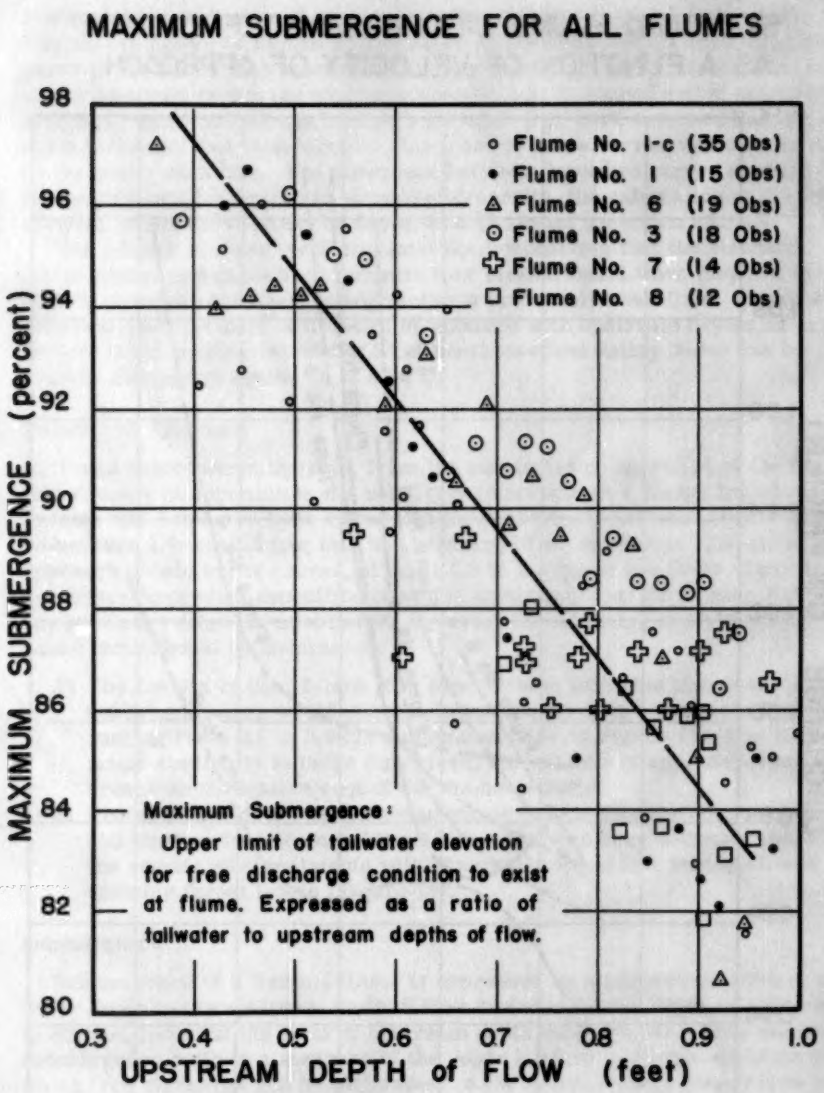

MAXIMUM SUBMERGENCE FOR ALL FLUMES
MAXIMUM SUBMERGENCE (percent)
98
96
94
92
90
88
86
84
82
80
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
UPSTREAM DEPTH of FLOW (feet)
Flume No. I-c (35 Obs)
Flume No. 1 (15 Obs)
Flume No. 6 (19 Obs)
Flume No. 3 (18 Obs)
Flume No. 7 (14 Obs)
Flume No. 8 (12 Obs)
Maximum Submergence:
Upper limit of tailwater elevation for free discharge condition to exist at flume. Expressed as a ratio of tailwater to upstream depths of flow.

Figure 12

When a flume is placed in a channel having a continuous slope through the measuring station, the fact that the tailwater may drown the flume (i.e., exceed the limit of maximum submergence) presents a real hazard. For this reason it is necessary to limit the normal depth of the maximum flow to be measured by the flume. Thus, if it is assumed that the maximum upstream depth d_1 is not to exceed 0.90 D and that the maximum recommended submergence is about 85% of this depth, then the normal depth of the flow in the channel should not exceed 0.75 D.

Channel Slope

Water surface measurements with Flume 1c both in and out of the channel were made through a wide range of upstream channel slopes in order to ascertain conformance of the flume to the energy criteria previously described. Figure 13 presents the calibration curves for the Flume 1c experiments with different upstream channel slopes, in terms of channel slope because "uniform" flow conditions could not be obtained and therefore the slope of the energy gradient was difficult to determine. At 2% slope, the calibration curve remained unchanged from that developed during the free discharge tests made under ideal conditions. When the channel slopes were greater than 3%, there was only a vague resemblance to the usual curve. At slopes greater than 3.0% the water became extremely turbulent. At 4.0%, 4.5%, and 5.0% no backup would occur; rather, one had to be formed artificially. Sometimes, shooting flow would wash the backup pond through the flume. No data were taken under the latter condition; hence, the discontinuance of the 4.5% and 5.0% curves.

This phenomena may be shown to conform with the theory developed for the flumes by the use of Figure 14. The three sets of curves shown on the figure are: (a) specific energy curves for the flows of 0.4, 0.8, 1.2, 1.6, and 2.0 cfs in the U-shaped upstream channel; (b) experimental "normal" depth curves for the channel slopes shown, determined on the basis of actual Q and channel depth at d_1; and (c) curves of $\frac{V^2}{2gd_n}$, i.e., lines of proportionality between velocity head and normal depth of flow in the channel.

When d_1 of Flume 1c for an indicated flow is plotted on its corresponding specific energy curve and a vertical line is projected downward from that point, the vertical projection will intersect the lower half of that same specific energy curve at the point indicated by a small circle. Several such points are plotted on Figure 14. These points lie approximately on the 2.0% slope line, indicating that the 2.0% channel slope is approximately the maximum at which Flume 1c will function properly throughout the range of flows represented. That such is the case may be observed in Figure 13.

Note that the circles described above lie slightly to the right of the $\frac{V^2}{2gd_n} = 2.0$ line indicating that Flume 1c can be expected to function properly in a channel where the velocity head does not exceed twice the normal depth of flow. Reference to the $\frac{V^2}{2gd_n} = 2.5$ line on Figure 14 will show that this line very nearly defines the upper limit of energy in the flow for which the flume may be expected to function properly, since the energy content is only slightly in excess of that needed to operate the flume.

EFFECT of SLOPING APPROACH CHANNEL

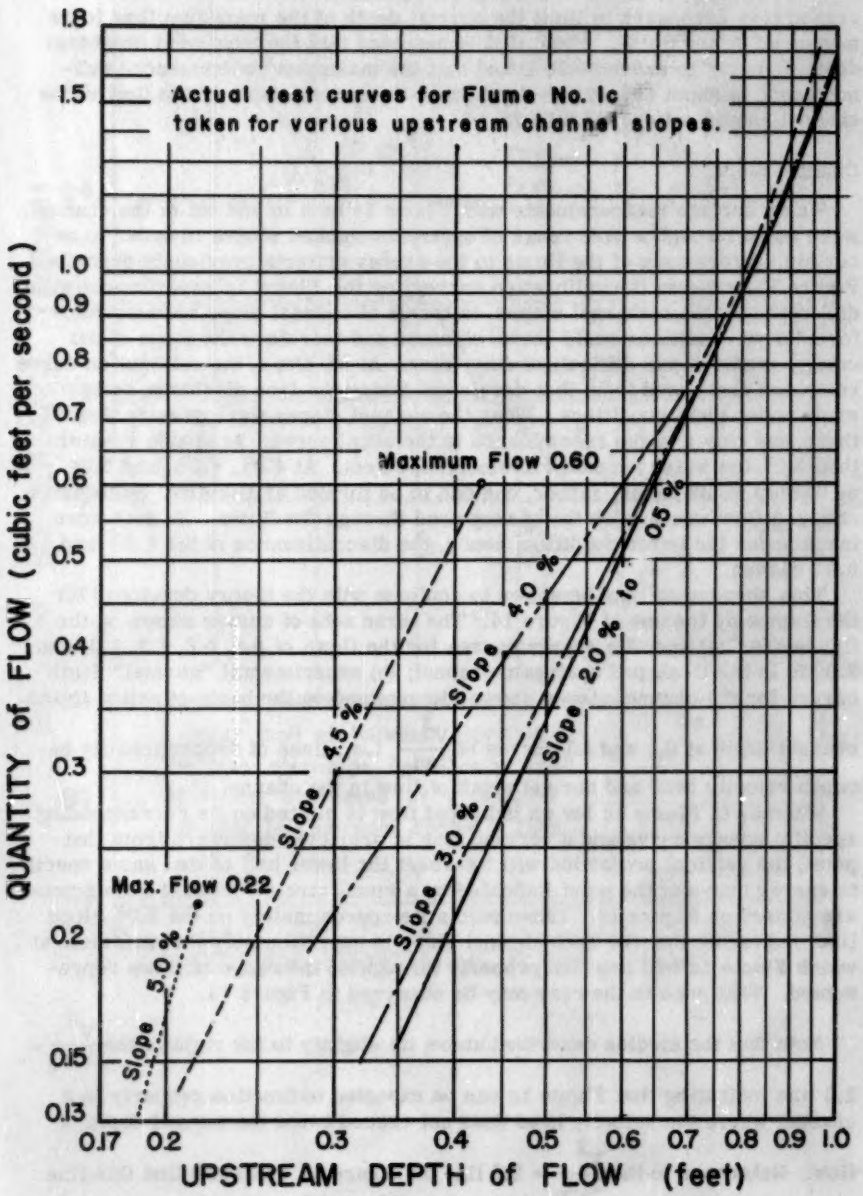


Figure 13

Further investigations with other standard flumes indicated that if $\frac{V^2}{2gd_n}$ is less than 1.5, operation of the flume will be satisfactory. Where the proportionality number exceeds 1.5 for channel flow at a proposed installation the proper limiting number should be determined by plotting an upstream depth d_1 for the given flume on Figure 14. Thus for the lower limit of Zone 3, Figure 2, $d_n = \frac{V^2}{3g}$ is arbitrarily assumed as the minimum value for design purposes. It is suggested that this check of energy content of normal channel flow be made on the maximum flow to be measured by the flume.

Throat Length

Six similar erection flumes (No. 1 series (in which throat length varied from D/8 (0.12 ft.) to 2 D (2.0 ft.) were investigated to determine the affect of throat length on the accuracy of flow measurement.

Figures 15 and 16 show the effect of throat length on the calibration curve of Flume 1c. Figure 17 demonstrates the influence of throat length on the coefficient of discharge for different depths of flow. For example, if two flumes with throat lengths of D/8 and D were used with an assumed discharge coefficient of 1.0, the indicated flow rate for the short flume might be as much as 30% less than the longer flume. The results of these tests indicate that the throat length should be approximately equal to the pipe diameter. It should in no case be less than about 0.60 D.

Side Slope

Flumes 1, 3, and 4 (Figures 4 and 7) had side slopes of 1:3, 1:2, and 1:1, respectively; Flume 6 (Figure 8) was a broad crested weir and had no sides as such but used the channel walls as part of the throat section. Observation of the flow characteristics and the similarity of the calibration curves indicates that the shape of the throat is relatively unimportant providing that it satisfies the necessary energy relationships.

The rating curves for Flumes 3, 4, and 6 were not continuous functions because the sides of the throat intersected the channel well below the top of the pipe. The fact that part of the throat is trapezoidal and part rectangular or circular merely complicates the calculations but otherwise does not impair the accuracy of the flume.

Base Height

Palmer and Bowlus[7] use D/8 and D/11 for the height of the base above the pipe invert while Ludwig and Ludwig[10] use D/12. Some of the English flumes[11] have no base; they rely upon the side constriction to force the flow through critical velocity. The base of a trapezoidal flume serves two functions: 1) it holds the sidewalls in place with respect to each other, and, 2) it aids in forming a plane-shaped throat which is convenient though not essential in rating the flume. It does not, as might be implied, add any safeguard against submergence of the flume that would not be accomplished by increasing the sidewall constriction.

The base height, t, is defined as the difference in elevation between the channel invert at the point of measurement of the upstream depth and the top

SPECIFIC ENERGY CHART SHOWING EXPERIMENTAL LIMIT OF CHANNEL SLOPE FOR FLUME Ic

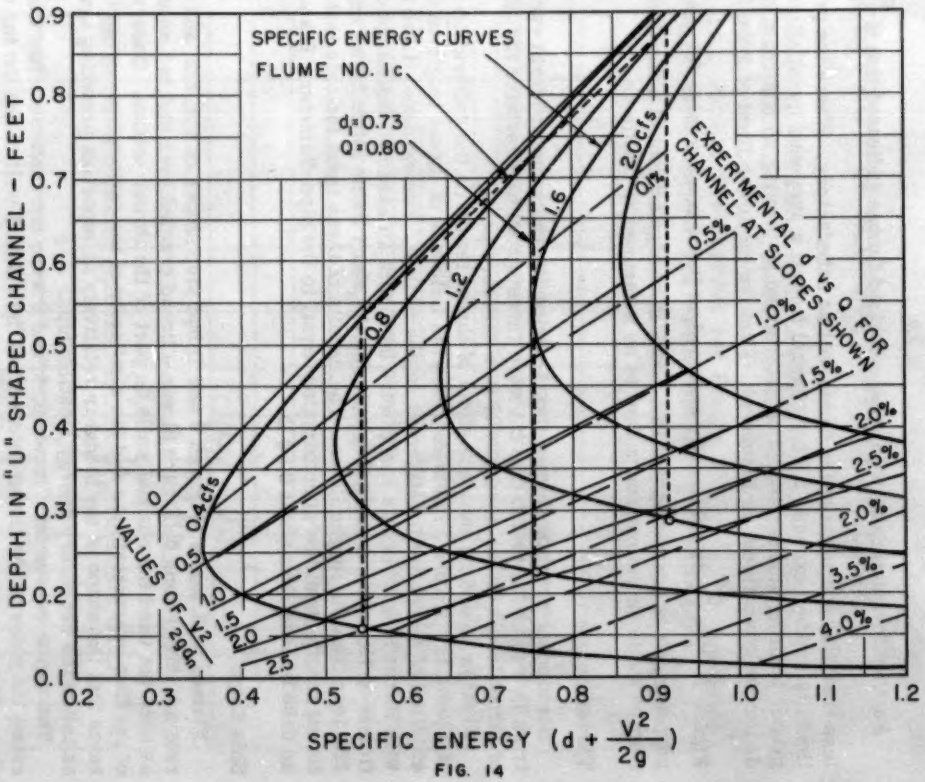


FIG. 14

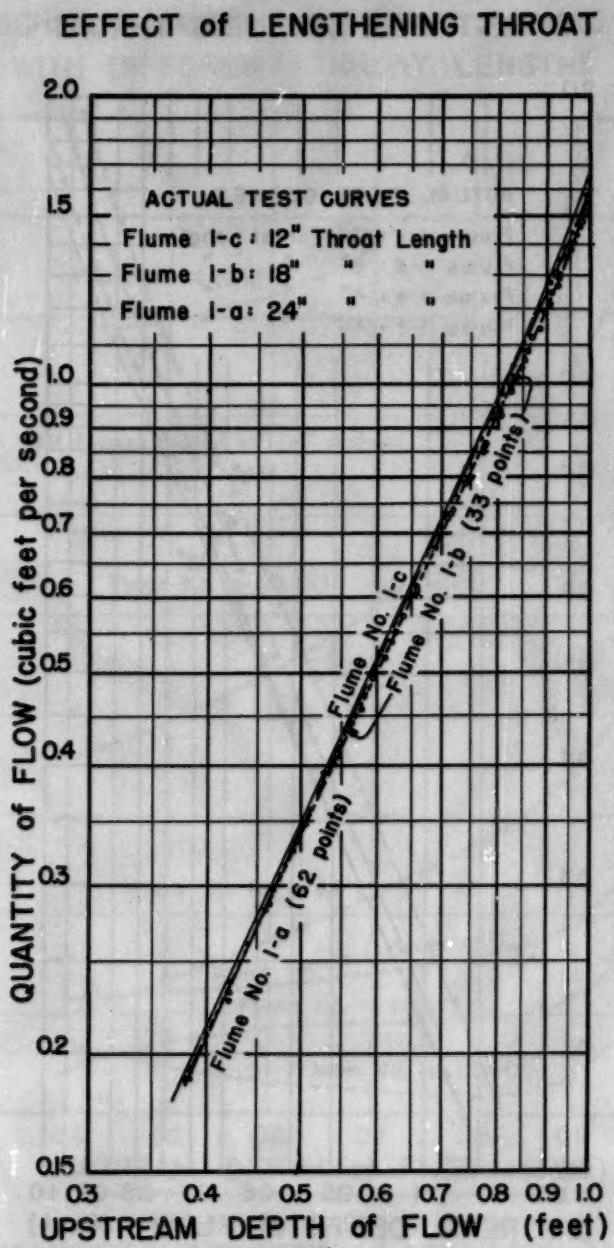

EFFECT of LENGTHENING THROAT
ACTUAL TEST CURVES
Flume I-c : 12" Throat Length
Flume I-b : 18" " "
Flume I-a : 24" " "
Flume No. I-c
Flume No. I-b (33 points)
Flume No. I-a (62 points)
QUANTITY of FLOW (cubic feet per second)
2.0
1.5
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.15
0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0
UPSTREAM DEPTH of FLOW (feet)

Figure 15

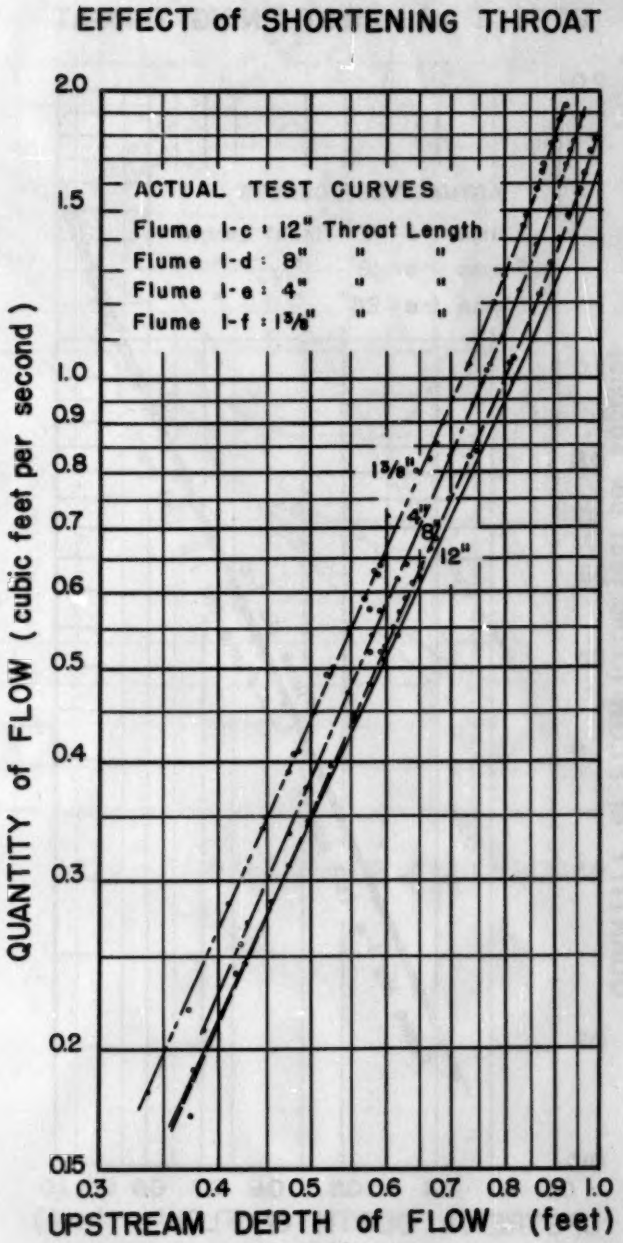

EFFECT of SHORTENING THROAT
ACTUAL TEST CURVES
Flume I-c : 12" Throat Length
Flume I-d : 8" " "
Flume I-e : 4" " "
Flume I-f : 1 3/8" " "
1 3/8"
4"
8"
12"
QUANTITY of FLOW (cubic feet per second)
2.0
1.5
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.15
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
UPSTREAM DEPTH of FLOW (feet)

Figure 16

DISCHARGE COEFFICIENTS FOR FLUMES WITH DIFFERENT THROAT LENGTHS

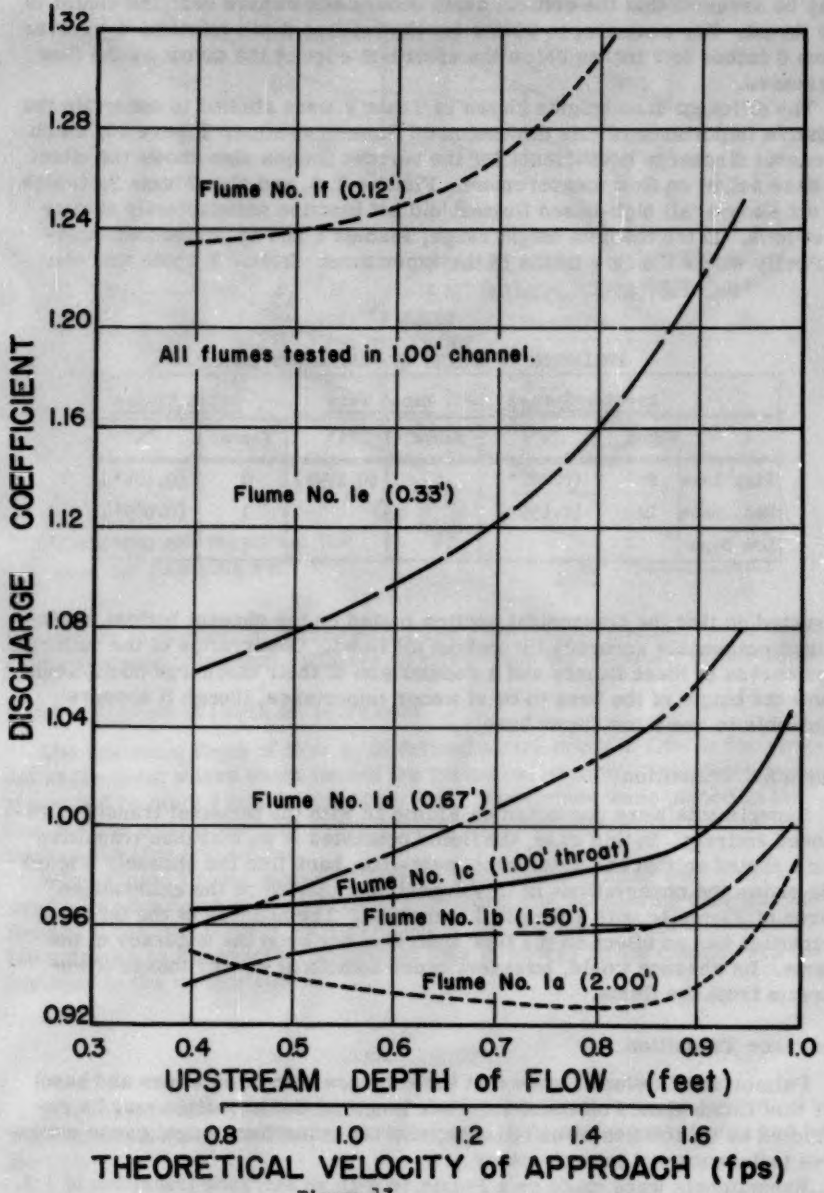


Figure 17

of the flume base at the point of occurrence of the critical depth. The flow in the throat of a Venturi flume is curve linear so that any determination of the point of critical depth is hypothetical. For practical purposes, however, it may be assumed that the critical depth occurs somewhere near the center of the throat. For example, in Flume 1c, the critical depth (vertical d_c) moves from 6 inches to 9 inches below the upstream edge of the throat as the flow increases.

The different base heights shown in Table V were studied to ascertain the relative importance of this dimension on flume operation. Figure 10, which presents discharge coefficients for the various flumes also shows the effect of base height on flow measurement. Flumes 8, 6, and also Flume 2c (which is not shown), all high-based flumes, did not function satisfactorily at very low flows. In the medium height range, Flumes 1 and 1c performed satisfactorily within the flow limits of the experiment. Flume 7 which was constructed so that the trapezoidal section rested on the channel bottom, maintained reasonable accuracy throughout all flows. Observation of the calibration curves of these flumes and a comparison of their discharge coefficients show the height of the base to be of minor importance, though it appears desirable to use a low flume base.

TABLE V

VARIATION IN HEIGHT OF BASE ABOVE INVERT

	Erector Series		Broad Weir		Solid Flumes	
	Flume	"t"	Flume	"t"	Flume	"t"
High Base	2c	(0.167')	6	(0.179')	8	(0.184')
Med. Base	1c	(0.050')			1	(0.089')
Low Base					7	(0.035')

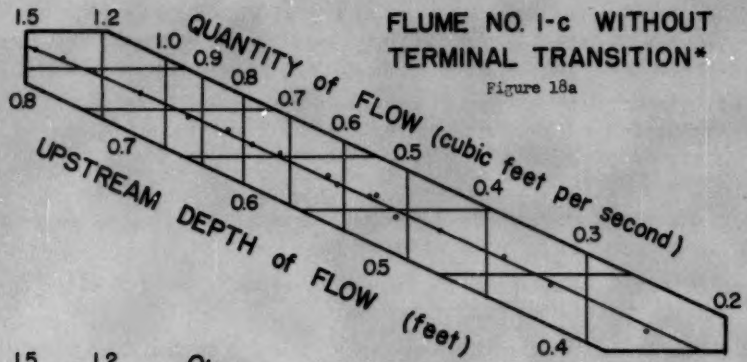
Terminal Transition

Experiments were conducted on Flume 1c with the terminal transition removed entirely. In this case, the flume consisted of an entrance transition and a throat so that the water shot, weir-like, back into the channel. Figure 18a shows the observations of this experiment plotted on the calibrations curve of Flume 1c with a terminal transition. The absence of the terminal transition has no effect on the flow upstream nor upon the accuracy of the flume. Its absence would, however, cause additional energy losses downstream from the flume.

Entrance Transition

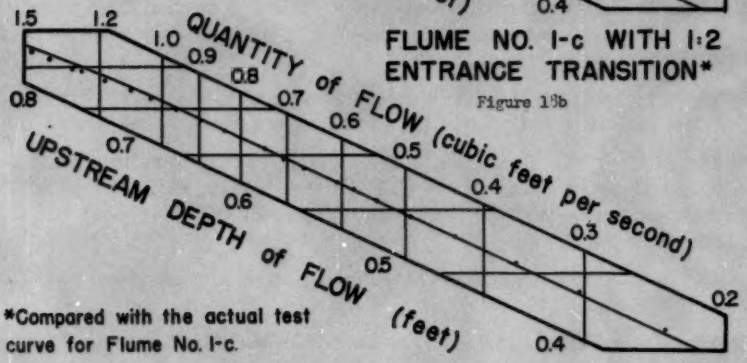
Palmer and Bowlus[7] state that the entrance transition (sides and base) for this flume should be about 1:3. The length of the transition may be restricted by difficulties of installation; shorter transitions might cause excessive turbulence and loss of energy.

Experiments were made on a Flume 1c with an entrance transition of 1:2. Figure 18b presents these observations plotted on the calibration curve of Flume 1c, with a 1:3 entrance transition. Since the difference between these



FLUME NO. I-c WITHOUT TERMINAL TRANSITION*

Figure 18a



FLUME NO. I-c WITH 1:2 ENTRANCE TRANSITION*

Figure 18b

*Compared with the actual test curve for Flume No. I-c.

two curves is negligible, it may be assumed that minor changes in the slope of the entrance transition will have no effect upon the accuracy of the flume.

Point of Upstream Depth Measurement

The upstream depth of flow d_1 is defined as the depth of flow in the channel at the point where water enters the transition section. This depth was measured by point gauge in a stilling well. The flumes were placed in the test section so that the port of the stilling well was adjacent to the leading edge of the side transitions.

When Flume 1c was moved downstream below the stilling well port, a small drawdown curve of increasing curvature with increasing flow was effected. Thus, the point of measurement of the upstream depth should be at the entrance to the flume or additional loss of energy in the channel will be included in the measurement.

CONCLUSIONS

The results of this investigation of the type of Venturi flume suggested by Palmer and Bowlus lead to the following conclusions about its design and behavior.

1) The Arredi method of rating a Venturi flume will give satisfactory results with a high degree of accuracy throughout a wide range of flows and channel conditions.

2) The coefficient of discharge varies from 0.97 to 1.03 as a function of depth of upstream flow for normal operation where the depth of upstream flow does not exceed 0.90 of the pipe diameter (0.90 D).

3) Standard flumes will function properly if the velocity head at depth for the maximum flow is not greater than 1.5 times the normal depth. For special cases, this ratio may be exceeded if proper investigation is made of the result.

4) The velocity of approach can be greater than 2.0 fps and is not relatively important as a design criterion.

5) The maximum submergence should not exceed 85 percent of the upstream depth at the maximum flow.

6) These flumes will measure up to 0.75 D under normal flow conditions, (i.e. 90% of the maximum pipe capacity), but may vary with the slope of the pipe and its roughness.

7) The throat length of 1.0 D is preferable, but it may vary from 0.6 D to 1.5 D without greatly affecting the accuracy of the flume.

8) The sides of the flume may slope as desired providing the flume is properly rated. Side slopes of 1:2 with a bottom width of D/3 permit the measurement of a wide range of flow (0.2 cfs to 1.75 cfs for 12" diameter pipe).

9) The height of the base for trapezoidal flumes should be between D/10 and D/20.

10) An entrance transition of 1:3 is desirable but small changes in the slope do not affect the accuracy of the flume.

11) The slope of the exit transition has no effect on the accuracy of measurement. Exit transitions are desirable only to conserve energy.

12) The point of upstream depth measurement should be no more than 0.5 D upstream from the entrance to the flume.

ACKNOWLEDGMENTS

The research on Palmer-Bowlus flumes was sponsored jointly by the Civil Engineering Division and the Sanitary Engineering Research Laboratory of the University of California at Berkeley, California.

Professors Joe W. Johnson and Erman A. Pearson assisted the authors in reviewing the analysis of the basic research. Special acknowledgment is made to Mr. Jayantilal Parikh for his helpful assistance in the conduct of the laboratory experiments.

Mr. Norman H. Anderson of the County Sanitation Districts of Los Angeles County encouraged the testing program and made many valuable suggestions regarding the practical installation problems of Venturi Flumes.

TABLE VI
RATING CURVE CALCULATIONS
FOR FLUME 1C.

$A = (y + \frac{d_c}{m}) d_c$

$B = y + \frac{2d_c}{m}$

$\frac{A}{2g} = \frac{V^2}{2g}$

$y = 0.337'$

$m = 3.20$

$t = 0.090'$

$D = 1.000'$

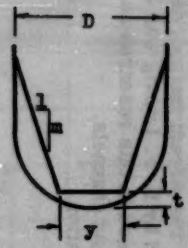


Figure 19

Table of Calculations

1) d_c	2) dt $= d_c + t$	3) $y + \frac{d_c}{m}$	4) $A = (y + \frac{d_c}{m})d_c$	5) $2B = 2y + \frac{4d_c}{m}$	6) $\frac{V^2}{2g} = \frac{A}{2B}$	7) d_p	8) A_p	9) Q	10) d_1
.10	.190	.368	.0368	.799	.0460	.10	.0408	0.2	0.378
.20	.290	.400	.0800	.924	.0866	.20	.1117	0.4	0.528
.30	.390	.431	.1293	1.049	.1232	.30	.1960	0.6	0.636
.40	.490	.462	.185	1.174	.1577	.40	.2930	0.8	0.727
.50	.590	.495	.2475	1.299	.1905	.50	.3925	1.0	0.807
.60	.690	.524	.3143	1.424	.221	.70	.5925	1.2	0.877
.70	.790	.556	.389	1.549	.251	.90	.7925	1.4	0.947

APPENDIX I

Rating Curve Calculations for Flume 1C

Any of several graphical or semi-graphical methods may be used to calculate the rating curve for a Venturi flume. The Arredi method, developed by Filippo Arredi[8] in 1936, and as clarified by H. F. Ludwig was used for calculating the rating curves of the flumes used in this research as it is easily adaptable to different shaped flumes and channels.

By means of the Arredi diagram, the equation

$$d_1 = d_c + t + \frac{A_c}{2B_c} - \frac{V_1^2}{2g}$$

is solved graphically for values of d_1 corresponding to assumed values of Q. To do this, three curves are plotted on the Arredi diagram, Figure 20: Curves showing area as a function of distance above the channel invert for the channel section and the throat section of the flume, and a curve of area as a function of velocity head for critical flow in the throat. The calculations for these curves are shown in Table VI.

Calculations for Table VI:

Col. 1 - Assume values of critical depth in throat.

Col. 2 - Add height of base (t) to obtain height of surface above pipe invert.

Col. 3-4 - Find area as $d_c\ (y + \frac{d_c}{m})$, plot on Arredi diagram as d_t vs A.

Col. 5 - Find 2B as $2y + \frac{4d_c}{m}$.

Col. 6 - Find $\frac{V^2}{2g} = \frac{A}{2B}$ for throat, plot $\frac{A}{2B}$ vs A on Arredi diagram.

Col. 7 - Assume depth in "U"-shaped pipe.

Col. 8 - Find Corresponding pipe Area (from standard handbooks) Section above 0.5' is rectangular. Plot d_p vs A on Arredi diagram.

Col. 9 - Assume value of rate of flow.

Col. 10 - From diagram scale value $(d_c + t + \frac{V^2}{2g})_t$ for a given flow rate (an example is shown for 0.4 cfs). With the above value, find the depth in the pipe (d_1) for which $(d_1 + \frac{V^2}{2g})_p = (d_c + t + \frac{V^2}{2g})_t$.

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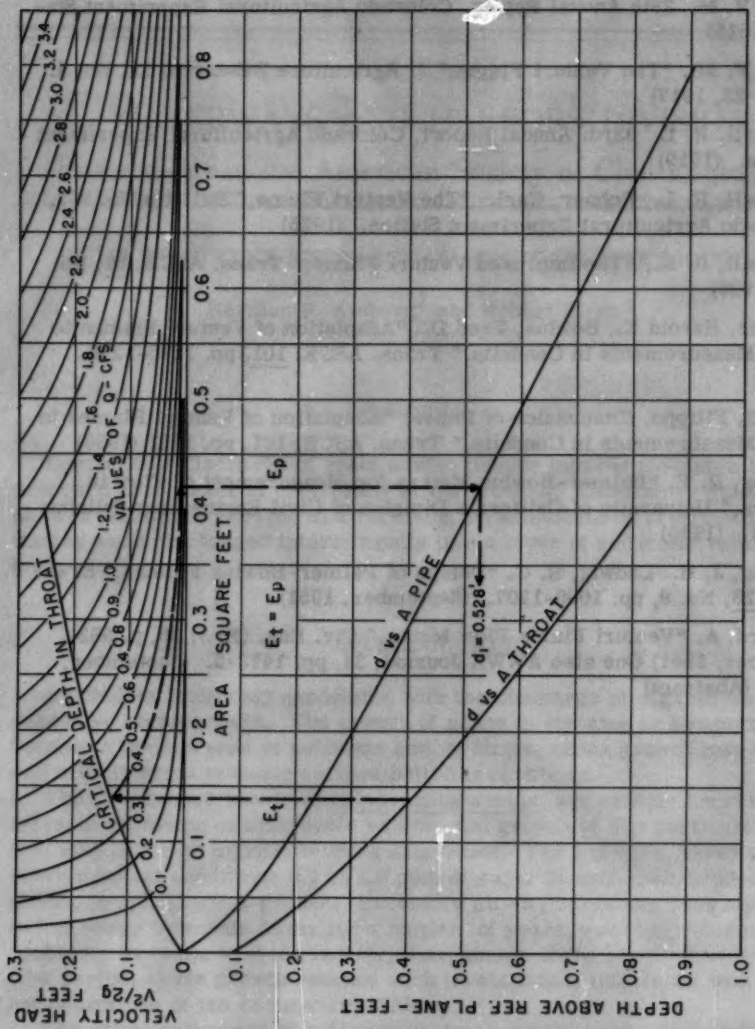
ARREDI DIAGRAM
FLUME No. Ic
VELOCITY HEAD V²/2g FEET
DEPTH ABOVE REF PLANE - FEET
AREA SQUARE FEET
CRITICAL DEPTH IN THROAT
VALUES OF Q - CFS
d vs A PIPE
d vs A THROAT
d₁ = 0.528'
Et = Ep
Et
Ep

Figure 20

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Paper 929 SA 2

JOURNAL
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

INTERMITTENT DISCHARGE OF SPENT SULFITE LIQUOR

Herman R. Amberg[1] and Robert Elder[1]
(Proc. Paper 929)

SYNOPSIS

Intermittent discharge of spent sulfite liquors into the Columbia River is proposed as a means of controlling slime growths. Laboratory studies showed that slime growths in a receiving stream could be controlled if the wastes were discharged intermittently into a river of sufficient volume and high velocity.

INTRODUCTION

A problem commonly associated with the discharge of organic wastes is excessive slime growth. The growth of slime in streams is a natural response to the increase in nutrients and, at times, slime growth may become sufficiently great to cause serious pollution problems.

The filamentous bacteria, Sphaerotilus natans, are usually found in streams receiving carbohydrate wastes, and growth of this particular organism may occur on extremely weak substrates. For example, under proper environmental conditions 0.5 to 1.0 ppm of sugar is sufficient to produce considerable sphaerotilus growth. Excessive slime growth has been a problem on the lower Columbia River for a number of years, and the problem will undoubtedly increase with the industrial expansion of the area. During the low flow period, slime growth reaches such levels that it interferes with the gill net operations of the commercial fishery.

The primary sources of pollution in the lower Columbia River are domestic sewage, spent sulfite liquor and numerous industrial wastes. The spent sulfite liquor from the production of about 500 tons of pulp per day is discharged into the lower Columbia River, or about 140 tons of wood sugar per operating day. Domestic sewage discharged in the critical area in

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conjunction with the carbohydrate load aggravates the problem by supplying sufficient organic nitrogen for the stimulation of excessive sphaerotilus growth.

Although the slime organism will develop on inorganic nitrogen, growth is stimulated by the addition of traces of organic nitrogen supplied by sewage. For example, Ruchhoft and Kachmer(1) found that Sphaerotilus natans had difficulty using glucose in a medium containing only glucose and mineral salts. In one experiment in a medium containing glucose and mineral salts, only 41 out of 1000 ppm of glucose originally present were used in 120 hours by a culture of Sphaerotilus natans. With settled sewage the organism was able to act upon 800 to 900 ppm of glucose within 24 to 48 hours after inoculation.

Experimentation was started in 1955 at the Engineering Experiment Station of Oregon State College to find a method of discharge which would eliminate or reduce the slime problem. This paper is a summarized account of the initial phase of investigation conducted to determine the effect of intermittent feeding upon the growth rate of Sphaerotilus natans.

Since bacteria require a constant supply of nutrients for optimum growth and reproduction, elimination of nutrients during the major portion of the logarithmic growth phase should result in a substantial decrease in slime growth. Application of intermittent feeding by the sulfite mills on the Columbia River would require storage facilities sufficient to hold the waste for 20 to 24 hours, with pumping facilities of sufficient capacity to discharge the spent sulfite liquor in two to four hours per day.

In the studies reported, a continuous flow apparatus was used to determine the effect of intermittent discharge of spent sulfite liquor upon sphaerotilus growth.

Apparatus

A diagram of a single unit of the study apparatus is shown in Figure 1. A rack of 12 columns was used in the study. Raw river water entered the constant head reservoir, where it was heated to 20°C by an immersion heater controlled by a thermoregulator. Raw waste was injected into the water line by a multi-feed constant displacement pump. The continuous water and waste flow entered the bottom of the column and was discharged at the top. Water flow was regulated by adjusting the height of the effluent line. A plasterers' lath was centered in the column to provide surface for the attached slime growths, and a recirculation pump having a capacity of 2 liters/minute was used to recirculate the water, ensuring sufficient velocity for slime growth.

Preliminary studies indicated that maximum growth was attained in seven days, and growth rapidly decreased after the seven-day growth period. At the end of seven days, the columns were disconnected and drained. The growth was carefully scraped from the column walls and the lath and collected in a two-liter graduate. The volume of growth was recorded after a 30-minute settling period. The settled growth was dried and total and volatile solids determinations were conducted on each sample. Because of the voluminous nature of sphaerotilus growth, the volume determinations are more indicative of nuisance conditions than the dry weights. The solids determinations are affected by adsorption of silt and debris and give poor indication of the amount of slime growth.

The growths collected from each column were subjected to a careful microscopic examination to determine the type of growth.

The fresh water flow through the columns was maintained at 500 ml/minute, and the water temperature ranged from 20 to 22°C.

Analyses of the ammonia and calcium base liquors used in these studies are presented in Table 1.

Spent sulfite liquor concentrations are recorded as ppm of bone dry solids in this report.

Experimental Results

A. Effect of Intermittent and Continuous Feeding of Ammonia Base Liquor Upon Slime Growth

The initial experiment was designed to demonstrate the difference between intermittent and continuous feeding of ammonia base liquor as measured by the amount of slime growth at the end of the seven-day test period.

Extensive surveys of the Columbia River below the pump mill outfalls have shown that maximum spent sulfite liquor a short distance below the pulp mill outfall is about 5 ppm. If the entire daily production of liquor were discharged over a two-hour period, the maximum concentration in the receiving stream would then be 60 ppm, and discharge over a four-hour period would give a maximum concentration of 30 ppm. These concentrations would decrease further as the waste progresses downstream.

The results of the initial experiment are presented in Table 2. Discharge of the spent sulfite liquor over periods of 2, 4, and 6 hours eliminated sphaerotilus growth entirely. Microscopic examination of the small amount of growth failed to show any signs of Sphaerotilus natans. As the feeding period was increased beyond six hours, sphaerotilus growth was again noted.

The striking difference in growth can readily be seen by comparing the volume of growth under continuous and intermittent feeding conditions. For example, 60 ppm of sulfite solids fed over a two-hour interval produced no visible growth, whereas the same amount of solids fed continuously (5 ppm) produced 800 ml of growth at the end of seven days. Intermittent feeding intervals up to six hours produced growth comparable on a volumetric basis to the "control," which received no spent sulfite solids.

The elimination of slime growth at the 60 ppm concentration was not caused by any toxic effect, since 60 ppm of spent sulfite solids fed continuously resulted in prolific growth (1950 ml).

B. Effect of Intermittent and Continuous Feeding of Calcium Base Liquor Upon Slime Growth

In the second series of experiments, calcium base liquor was substituted for the ammonia base liquor. The results of the second experiment are presented in Table 3.

No significant difference in slime growth could be detected in columns which were fed intermittently and the "control" columns. Again, microscopic examination of the small amount of growth showed no Sphaerotilus natans in any of the columns receiving intermittent doses of spent sulfite solids.

Continuous feeding of equivalent amounts of calcium base spent liquor gave considerable growth. Microscopic examination of the growth developed under continuous feeding showed that the growth was almost a pure culture of Sphaerotilus natans.

C. Effect of an Ammonia Base Residual Upon Slime Growth, Employing Intermittent Feeding

Since collection of spent liquor from the blow pits is not complete, it was thought of practical interest to determine the effect of a continuous low concentration of spent sulfite solids upon slime growth when employing an intermittent feeding schedule. A background concentration of 2 ppm spent sulfite solids was fed to the column on a continuous basis and the intermittent feeding schedule of the initial experiment was repeated.

Intermittent feeding on a two-hour basis produced no significant difference in slime growth when compared with the control and the column receiving a continuous concentration of 2 ppm (Table 4). Increasing the intermittent feed interval to four to six hours resulted in some sphaerotilus growth, but growth was considerably less than under a continuous feeding schedule.

All samples of slime from the columns receiving a 2 ppm spent sulfite solids residual (continuous feed) showed the presence of Sphaerotilus natans under careful microscopic examination.

D. Effect of Two Intermittent Feedings Per Day Upon Slime Growth

Two intermittent feedings per day would have definite advantages over the single feeding, since less storage capacity and smaller pumps would be required to handle the spent liquor.

In this experiment two daily feedings were employed, one at two-hour intervals twice a day, and the other at three-hour intervals twice a day. One series received a continuous discharge of 2 ppm ammonia base solids in addition to the intermittent discharges.

Referring to Table 5, it can be seen that two intermittent feedings were very effective in eliminating slime growth, provided the water carried no continuous residual. In the presence of a continuous residual of 2 ppm, growth did develop, although it was much less than under a continuous discharge schedule.

DISCUSSION

All of the laboratory results presented indicate that sphaerotilus growth can be eliminated or reduced by modification of the method of discharge at the mill. Results indicate that optimum growth occurs when spent sulfite liquor is discharged continuously at a constant concentration. Through the use of an intermittent feeding schedule, the same amount of spent liquor can be discharged without the subsequent increase in slime growth.

Under an intermittent feeding schedule, the organism is unable to adjust or become acclimated to any one condition. The intermittent schedule also eliminates a major portion of the reproduction cycle during the phase of logarithmic growth. Although intermittent feeding to water containing a spent sulfite liquor residual resulted in some growth, the growth was substantially lower than if the same quantity of waste were discharged over a 24-hour period.

Tables 2, 3, and 4 clearly demonstrate the effectiveness of an intermittent feeding schedule. Since there is always a possibility of some equalization upon discharge to a swift stream, it becomes advantageous to discharge over as short a period as possible. Intermittent discharge is effective in reducing

slime growth, provided the growth is attached and does not grow in the flowing portion of the stream, and provided that the velocity is sufficiently swift to prevent equalization of the intermittent discharges. The receiving stream must also have sufficient flow to absorb the intermittent discharges without depleting the oxygen supply and without creating a toxicity problem.

The experimental data indicate that the period of discharge can be increased up to six hours without serious sliming, but if equalization should increase the discharge period upon downstream translation in excess of six hours, a slime problem may develop. Flow in the Columbia River is swift, and it is doubtful whether merging of the intermittent discharges would ever occur.

The intermittent discharge principle has considerable flexibility. For example, if the possibility of some equalization of discharges exists, two days' production over four hours, the possibility of developing a continuous discharge pattern in the lower river would be eliminated.

The possibility of a toxicity problem should be considered also. Voluminous survey data collected below the outfalls of the sulfite mills on the Columbia River, have indicated maximum spent sulfite liquor concentrations in the immediate vicinity of the underwater outfalls to be about 3.0 to 5.0 ppm of bone dry solids when discharging over a 24-hour period. Discharge of the entire day's production over a two-hour interval would result in a maximum concentration below the outfall of 24/2 x 5.0, or 60 ppm. Bioassays conducted indicate that these concentrations would not constitute a serious toxicity problem. Although the fish may be exposed to concentrations as high as 60 ppm, the time of exposure would be relatively short (2 to 4 hours), and this concentration would rapidly decrease upon passage downstream. For example, on the basis of available survey data, a discharge of 60 ppm at Camas would be reduced to less than 20 ppm at the Interstate Bridge at Vancouver.

Williams, et al[2] reported on the toxicity of spent sulfite liquor to salmonoid fishes tested under experimental conditions. Twenty long-term experiments (30 days) were conducted with Chinook, pink, and silver salmon of various age groups, obtained from hatcheries. From seven long-term experiments the following mortality "threshold indices" (5% mortality) were determined.

Fish	Age of Fish	ppm of Spent Sulfite Solids
A. Chinook salmon	38	117.5
	280	60.0
	305	56.0
B. Pink salmon	102	155.0
	198	53.0
C. Silver salmon	351	123.0
	412	101.5

Under an intermittent discharge schedule, the test specimens could undoubtedly tolerate much higher concentrations.

Engineering Implications

The possible utilization of intermittent feeding of organic wastes as a

means of waste disposal probably has limited application. The stream utilized must have sufficient flow to handle the slug loadings to prevent toxicity and oxygen depletion problems.

However, the method may be successfully used in some locations to shift the critical dissolved oxygen deficit or to lengthen the period of self-purification. It would be particularly applicable to streams where the major portion of self-purification occurs through the action of slime growths on the river bottom. By eliminating the slime growths, the rate parameter "k" would be reduced and the waste may be shifted from a critical stream to a larger body of water where the purification capacity would be adequate to handle the waste.

The discharge of nitrogen deficient wastes on an intermittent feeding schedule would also be effective in reducing the rate of decomposition. For example, if we assume a carbon to nitrogen ratio in a stream receiving an organic waste of 50 to 1, then by discharging the waste over a four-hour period the carbon to nitrogen ratio would be increased to 300 to 1, with a subsequent reduction in the rate of decomposition.

CONCLUSIONS

The experimental results presented warrant the following conclusions:

1. The most favorable feeding schedule for optimum growth of Sphaerotilus natans is a continuous discharge of spent sulfite liquor.

2. The intermittent discharge of spent sulfite liquor to river water having a low pollution residual produced no slime growth, provided the discharge interval was maintained below six hours.

3. The intermittent discharge of spent sulfite liquor to river water containing a background residual of 2 ppm of sulfite solids produced some slime growth, but the growth was much less compared with the continuous discharge of an equivalent pollution load.

SUMMARY

This report is a summarized account of the initial studies of the research program being conducted by the National Council for Stream Improvement at the Engineering Experiment Station of Oregon State College to study the factors affecting slime growth in streams receiving pulp mill wastes.

The initial experimentation presents a novel method of waste disposal designed to eliminate or reduce the amount of slime growth in the receiving stream. Since bacteria require a constant supply of nutrients for optimum growth and reproduction, elimination of nutrients during the major portion of the logarithmic growth phase should result in a substantial decrease in slime growth. Application of intermittent feeding by the sulfite mills on the Columbia River would require storage facilities sufficient to hold the waste for 20 to 24 hours, with pumping facilities of sufficient capacity to discharge the spent sulfite liquor in two to four hours per day.

A continuous flow apparatus was designed to afford sufficient velocity for optimum growth of slime (Sphaerotilus natans). The growth was removed from the test columns after seven days and measured.

The laboratory experiments showed that the most favorable method of feeding for optimum growth of Sphaerotilus natans was a continuous discharge of spent sulfite liquor. Intermittent discharge of spent sulfite liquor to river water having a low pollution residual produced no slime growth, provided the discharge interval was maintained below six hours. Intermittent discharge of spent sulfite liquor to river water containing a background residual of 2 ppm of sulfite solids produced some slime growth, but the growth was much less compared with the continuous discharge of an equivalent pollution load.

Although intermittent feeding of spent sulfite liquor is effective in eliminating attached slime growths, the application of this method of discharge is limited to streams of sufficient water volume and relatively high velocity. Dilution must be sufficient to prevent oxygen depletion and toxicity, and the velocity must be sufficient to prevent equalization of the intermittent discharges. Since a certain amount of equalization is inevitable, the period of discharge should be as short as possible, and the holding period should be as long as possible.

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Table 1.
Analyses of Spent Sulfite Liquors Used in Tests

Tests	Ammonia Base	Calcium Base
Total solids, %	11.6	13.8
Volatile solids, %	89.1	---
BOD (5-day at 20°C), ppm	30,000	39,000

Table 2.
Effect of Intermittent and Continuous Feeding
of Ammonia Base Spent Liquor Upon Slime Growth
Water Temperature 21°C - Time of Run 7 Days

SSL Concentration ppm Bone Dry Solids	BOD Added ppm	Feeding Interval hr	Resting Interval hr	Slime Volume ml
60	15.5	2	22	0
30	7.7	4	20	30
20	5.1	6	18	50
15	3.9	8	16	720
10	2.6	12	12	205
Control	0.5-1.0	-	--	50
5	1.3	24	--	800
10	2.6	24	--	1240
60	15.5	24	--	1950

Table 3.
Effect of Intermittent and Continuous Feeding
of Calcium Base Liquor Upon Slime Growth
Water Temperature 21°C - Time of Run 7 Days

SSL Concentration ppm Bone Dry Solids	BOD Added ppm	Feeding Interval hr	Resting Interval hr	Slime Volume ml
60	16.6	2	22	40
30	8.3	4	20	60
20	5.5	6	18	60
15	4.1	8	16	60
Control	0.5-1.0	24	--	40
5	1.4	24	--	550
10	2.8	24	--	840
15	4.1	24	--	730

Table 4.
Effect of Ammonia Base Residual Upon Slime Growth
Employing Intermittent Feeding

SSL Concentration Added, ppm		BOD Added ppm		Feeding Interval	Resting Interval	Slime Volume
Cont.	Intermit.	Cont.	Intermit.	hr	hr	ml
2	60	0.5	15.5	2	22	75
2	30	0.5	7.7	4	20	200
2	20	0.5	5.1	6	18	230
2	15	0.5	3.9	8	16	380
2	10	0.5	2.6	12	12	420
Control	--	0.5-1.0	--	--	--	50
2	--	0.5	--	24	--	110

Table 5.
Effect of Two Daily Intermittent Feedings of
Ammonia Base Liquor Upon Slime Growth

SSL Concentration Added, ppm		BOD Added ppm		Feeding Interval	Slime Volume
Cont.	Intermit.	Cont.	Intermit.	hr	ml
2	60	0.5	15.5	2 hr 2 × a day	300
0	60	0.0	15.5	2 hr 2 × a day	50
2	40	0.5	10.2	3 hr 2 × a day	590
0	40	0.0	10.2	3 hr 2 × a day	130

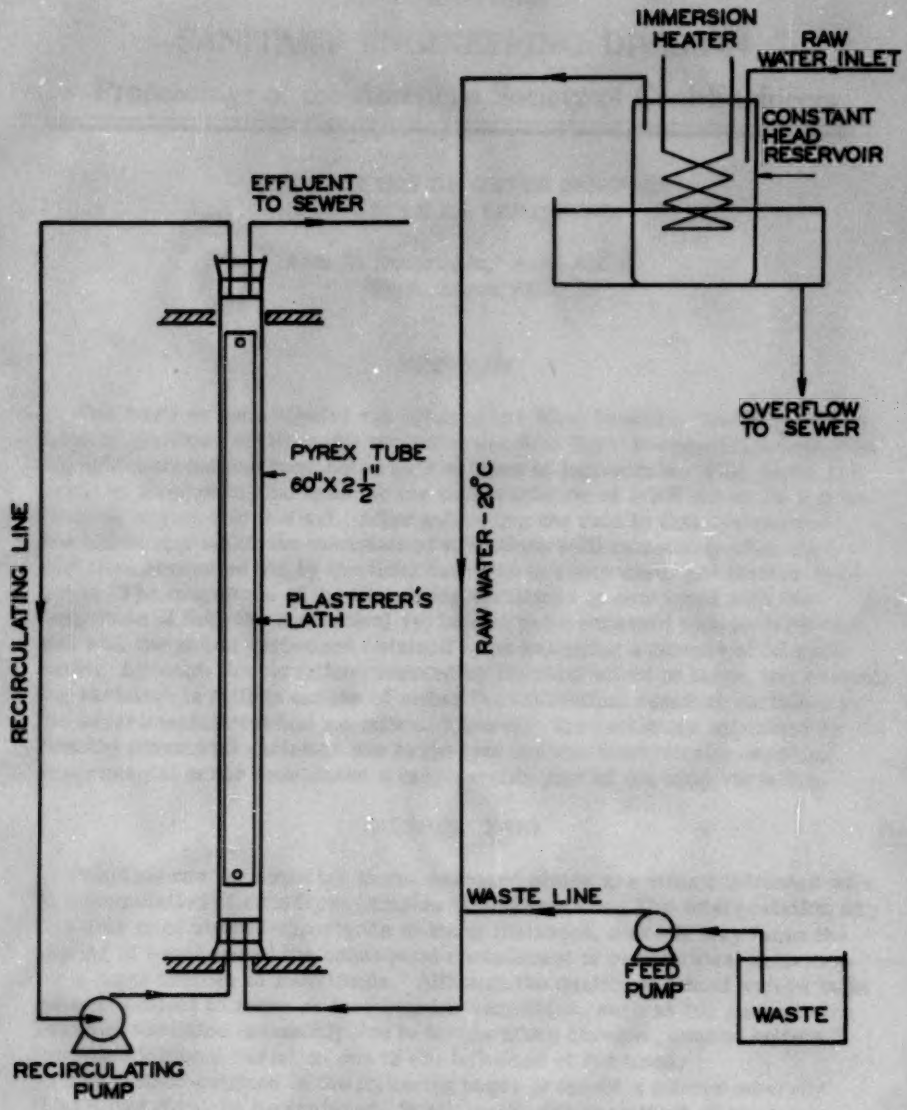
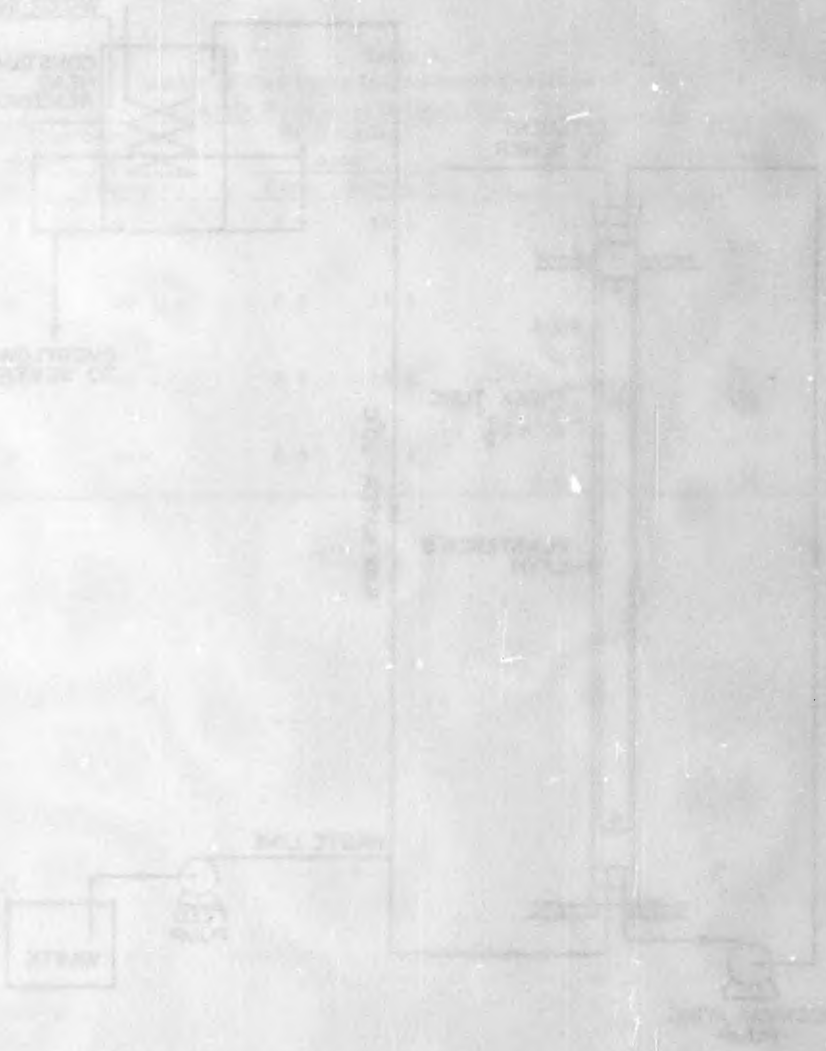


Fig. 1. Continuous Flow Apparatus for Sphaerotilus Growth Studies.



Paper 930 SA 2

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THE ANALYSIS OF WATER SAMPLES FOR CYCLICAL VARIATIONS

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SUMMARY

The more or less regular variation of the Most Probable Number of coliform organisms (MPN) in the waters in the New York Metropolitan Area with the tidal current has been noted by a number of individuals. This paper presents an attempt to characterize the tidal variation of MPN values by a quantitative, systematic method. After subjecting the data to this systematic method of appraisal, the unexplained variations still remaining after the variation accounted for by the tidal currents is subtracted, are further analyzed. The magnitude of the remaining variations is compared with the magnitude of both the theoretical variations to be expected with an MPN test, and with the actual variations obtained when analyzing a sample of 20 replicates. Although the variation removed by the tidal effect is large, the remaining variation is still in excess of either the theoretical residual variation or the experimental residual variation. However, the variations explained by both the sinusoidal variation due to the tide and the theoretically expected experimental error constitutes a considerable part of the total variation.

INTRODUCTION

Pollution control agencies in the seaboard states are often confronted with an accumulation of data from samples of tidal waters. The interpretation of this data is of utmost importance in many instances, since it may mean the closing of beaches and the consequent curtailment of recreational activities for a large number of individuals. Although the quality of inland waters is in general subject to more or less regular variations, such as for example the seasonal variation ostensibly due to temperature changes, coastal waters have an additional variation due to the influence of the tides.

The method outlined in the following pages presents a method whereby tidal water data can be analyzed. Specifically, the magnitude of the tidal

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influence and its phase relation to the tidal current can be determined, and the residual variation, whose magnitude determines to some extent the amount of tidal mixing, can be analyzed.

Although the method is presented for the analysis of tidal variations, any cyclical variations, or combination of cyclical variations, can be analyzed. As a matter of historical note, the general method was developed for the simultaneous analysis of seasonal and tidal variations in the waters of Upper New York Harbor.

Procedure

A sinusoidal type of curve was chosen as most likely to represent tidal variations in MPN concentrations inasmuch as both the tide (height of water) and the tidal current (horizontal movement of water) exhibit sinusoidal characteristics in New York Harbor.

The curve given in (1) below was chosen.

$$(1)\quad Y = K_1 + K_2 \sin\frac{t-t_o}{p}$$

in which

$Y = \log_{10}\text{MPN}$

K_1, K_2 and t_o are constants

p = the period which was chosen as the period of tidal cycle in New York Harbor equal to about 12.38 hrs.

The "least squares" method was used to evaluate the constants K_1, K_2 and t_o. In the form shown in (1) above the constants cannot be readily determined. However, since

$$\sin\frac{t-t_o}{p} = \sin\frac{t}{p}\cos\frac{t_o}{p} - \cos\frac{t}{p}\sin\frac{t_o}{p}$$

which formula is the familiar subtraction formula from trigonometry, we obtain by substitution

$$(2)\quad Y = K_1 + K_2\left(\sin\frac{t}{p}\cos\frac{t_o}{p} - \cos\frac{t}{p}\sin\frac{t_o}{p}\right)$$

from which we obtain

$$(3)\quad Y = K_1 + K_3 \sin\frac{t}{p} + K_4 \cos\frac{t}{p}$$

if we let

$$K_3 = K_2 \cos\frac{t_o}{p}$$

$$K_4 = -K_2 \sin\frac{t_o}{p}$$

Equation (3) is then capable of being used with respect to the observed data. By summing the observations and obtaining the partial derivatives of the summer equations with respect to the constants K_1, K_3 and K_4 one obtains the three normal equations

$$(4)\quad K_1 N + K_3 \Sigma \sin\tfrac{t}{p} + K_4 \Sigma \cos\tfrac{t}{p} = \Sigma Y$$

$$(5)\quad K_1 \Sigma \sin\tfrac{t}{p} + K_3 \Sigma \sin^2\tfrac{t}{p} + K_4 \Sigma \sin\tfrac{t}{p}\cos\tfrac{t}{p} = \Sigma Y \sin\tfrac{t}{p}$$

$$(6)\quad K_1 \Sigma \cos\tfrac{t}{p} + K_3 \sin\tfrac{t}{p}\cos\tfrac{t}{p} + K_4 \Sigma \cos^2\tfrac{t}{p} = \Sigma Y \cos\tfrac{t}{p}$$

These equations can be written in the matrix form as

$$(7)\quad \begin{bmatrix} N & \Sigma\sin\frac{t}{p} & \Sigma\cos\frac{t}{p} \\ \Sigma\sin\frac{t}{p} & \Sigma\sin^2\frac{t}{p} & \Sigma\sin\frac{t}{p}\cos\frac{t}{p} \\ \Sigma\cos\frac{t}{p} & \Sigma\sin\frac{t}{p}\cos\frac{t}{p} & \Sigma\cos^2\frac{t}{p} \end{bmatrix} \begin{bmatrix} K_1 \\ K_2 \\ K_3 \end{bmatrix} = \begin{bmatrix} \Sigma Y \\ \Sigma Y \sin\frac{t}{p} \\ \Sigma Y \cos\frac{t}{p} \end{bmatrix}$$

and if we let the matrix involving the sum of the dependent terms on the left equal to $[A_i]$ a simple form is obtained as follows below.

$$(8)\quad [A_i] \begin{bmatrix} K_1 \\ K_2 \\ K_3 \end{bmatrix} = \begin{bmatrix} \Sigma Y \\ \Sigma Y \sin\frac{t}{p} \\ \Sigma Y \cos\frac{t}{p} \end{bmatrix}$$

It is obvious that if the inverse of the matrix, $[A_i]^{-1}$, can be found, then by pre-multiplying the previous equation by the inverse of the matrix one obtains

$$(9)\quad [A_i]^{-1}[A_i] \begin{bmatrix} K_1 \\ K_2 \\ K_3 \end{bmatrix} = \begin{bmatrix} K_1 \\ K_3 \\ K_4 \end{bmatrix} = [A_i] \begin{bmatrix} \Sigma Y \\ \Sigma Y \sin\frac{t}{p} \\ \Sigma Y \cos\frac{t}{p} \end{bmatrix}$$

from which the solution of the constants are then readily obtained.

The matrix $[A_i]$ closely approximates a diagonal matrix in many instances, since the terms

$$\Sigma \sin\tfrac{t}{p}\,,\ \Sigma \cos\tfrac{t}{p}\,,\ \Sigma \sin\tfrac{t}{p}\cos\tfrac{t}{p}$$

are usually relatively small with respect to

$$N, \Sigma \sin^2 \frac{t}{p}, \Sigma \cos^2 \frac{t}{p}$$

the inverse can be expressed simply, as

$$(10) \quad [A_i]^{-1} = \begin{bmatrix} \frac{1}{N} & 0 & 0 \\ 0 & \Sigma \sin^2 \frac{t}{p} & 0 \\ 0 & 0 & \Sigma \cos^2 \frac{t}{p} \end{bmatrix}$$

from which it is apparent that the solution of the constants are then

$$(11) \quad K_1 = \Sigma Y / N$$

$$(12) \quad K_3 = \Sigma Y \sin \frac{t}{p} / \Sigma \sin^2 \frac{t}{p}$$

$$(13) \quad K_4 = \Sigma Y \cos \frac{t}{p} / \Sigma \cos^2 \frac{t}{p}$$

If sampling is carried on for other than a complete cycle, or more frequently during one part of the cycle, then the matrix is not diagonal and the solution of the three normal equations (4), (5) and (6) has to be carried out by other means such as the use of determinants.

Once the values of K_1, K_3 and K_4 have been obtained a value for K_2 and t_o can be calculated and then the general prediction equation (1) obtained. t_o and K_2 can be obtained from K_3 and K_4 by the use of

$$K_3 = K_2 \cos \frac{t_o}{p}$$

$$K_4 = -K_2 \sin \frac{t_o}{p}$$

The physical concept of the various constants can be obtained to some extent from Figure I. t is the elapsed time from an arbitrarily chosen starting point, the time at which the last maximum ebb current occurred. t_o represents the time at which the sinusoidal code starts, or in other terms, the phase angle. p of course is the period of the cycle which as previously stated is equal to about 12.4 hours. The amplitude, which is the maximum displacement from the mean value, is equal to K_2.

Analysis of Data

The observed values of MPN's used for this analysis were obtained at a sampling point in New York Harbor. As can be seen from Figure II samples were collected every 20 minutes for a 14 hour period. However, data for only the first 12 hours and 35 minutes were used in the analysis in order not to emphasize any one phase of the tidal cycle. Table I gives observed and derived data concerning these samples. The sum of some of the columns in the table are indicated below.

$\Sigma X_1 = +1.2407$ $\Sigma X_1X_2 = +0.09358751$

$\Sigma X_1^2 = +19.91224087$ $\Sigma X_1Y = +12.84269021$

$\Sigma X_2 = -0.5996$ $\Sigma X_2Y = +1.17102169$

$\Sigma X_2^2 = +18.07751444$ $N = 38$

$\Sigma Y = +154.5207$ where $X_1 = \sin \frac{t}{p}$

$\Sigma Y^2 = +639.62970095$ $X_2 = \cos \frac{t}{p}$

(The fact that $\Sigma \sin^2 \frac{t}{p} + \Sigma \cos^2 \frac{t}{p}$ should be equal to N can be used as a check of the computations.) The problem then exists of finding the inverse of the matrix shown following.

$$\begin{bmatrix} 38. & 1.2407 & -0.5996 \\ 1.2407 & 19.91224087 & 0.09358751 \\ -0.5996 & 0.09358751 & 18.07751444 \end{bmatrix}$$

It is evident that the values in other than the principle diagonal are small, hence, the approximation previously stated can be used in deriving the values of K_1, K_3, and K_4. They are as shown below.

$K_1 = +4.06633 = 154.5207/38$

$K_3 = +0.64496 = 12.8426902/19.91224087$

$K_4 = +0.06478 = 1.17102169/18.07751444$

A prediction equation such as (3) or one similar to (1) can be used. To obtain the K_2 and t_0 values for use with (1), the procedure described previously can be used as shown below.

$K_2 \cos X = +0.64496$

$K_2 \sin X = -0.06478$

$\cot X = -9.06478$

$X = -5.7^o = +354.3^o$

$t_0 = 12.18$ hours or -0.20 hours

$K_2 = \frac{.64496}{.99506} = 0.648$ use 0.65

$K_2 = \frac{-.06478}{-.09932} = +0.652$

The fact that the t_0 value is comparatively small (-0.20 hours) as compared to the length of the current period (12.38 hours) indicates that the current curve is approximately 90^o out of phase with the MPN curve as shown in Figure III.

This indicates that the MPN values reach their maximum at the end of the ebb current (ebb current slack). Upon the onset of the flood current, the

MPN values continue to drop until the end of the flood current period at which time the minimum MPN values are reached. When the ebb current begins, values then continue to rise again to their maximum value at the end of the ebb current. It appears that high MPN values are associated with the ebb currents.

For computational purposes (in order to draw a graph of the curve), equation (1) or (3) can be used. For this analysis equation (3) has been used.

An analysis of the differences between the values predicted by the sinusoidal equation (3) and the actual observed values was made in order to determine whether these differences could be considered as normal experimental errors.

The accompanying table, "Analysis of Residual Errors" presents the details of the analysis of the residual error. As the table stands, the residual errors do not form a normal distribution. This can be readily ascertained from the value of chi square of 103.86, which value of chi square is theoretically obtained less than 1% of the times were the distribution of residual errors a normal distribution. However, the high value of chi square is almost entirely due to the MPN observation which occurred at 1440 o'clock that has a value of 350,000 per 100 ml. If this value is not considered, then the remaining values of the residual errors are grouped normally. Inasmuch as the relatively high value of 350,000 may be due to "streaky" pollution, the remaining analysis is conducted after the deletion of this maximum value.

The re-computed values (after deleting the 350,000 figure) for the variance and standard deviation are as follows:

$$\sigma_o^2 = 0.167$$

$$\sigma_o = 0.408$$

If this observed value of the variance were to compare favorably with the combined experimental and theoretical variance of the MPN test, then there is a more or less complete explanation of all observed values. W. G. Cochran* presents a theoretical value for the variance and standard deviation as follows:

$$\sigma_t^2 = 0.067081$$

$$\sigma_t = 0.259$$

It is obvious that the observed variance is approximately 2-1/2 times that of the theoretically expected variance. However, the theoretical variance does not include the so-called "human" error. Any given series of observations would, of course, include both the theoretical error and the experimental or human error.

A set of 20 replicates of a sample (Table III) were used in order to estimate the total of the experimental and theoretical errors. An analysis of these replicates indicates that the variance and standard deviation of the sample are as follows:

---

* W. G. Cochran - "Estimation of Bacterial Densities by Means of the Most Probable Number," Biometrics 6:105-116 (June 1950).

$$\sigma_e^2 = 0.0909$$

$$\sigma_e = 0.3015$$

Although the variance obtained, 0.0909, of this combined experimental and theoretical error is still smaller than that obtained for the observed residual variance, 0.167, the major part of observed residual variance can be accounted for by the combined experimental and theoretical error.

A quantitative method to determine whether a sinusoidal curve is a reasonable representation of the original data is outlined in Table IV. For the example used, there is a significant reduction obtained in the total variation by the use of a sine curve. The residual mean square value shown in the Table IV, 0.253,608. should check the value of the variance obtained in Table II. The method of Table IV to obtain an estimate of the residual error is preferable to that of Table II since there is considerably less labor involved.

The sinusoidal variation which accounted for much of the total variation in MPN values in the example analyzed can be conceived as representing varying mixtures of two "reservoirs" of water each with different concentrations of coliform organisms. The "reservoir" with the high coliform concentrations contributes most of the water at the end of the ebb period. MPN values begin to lower as the high MPN "reservoir" waters are diluted by the low MPN reservoir waters, until the time of lowest MPN counts when the low MPN "reservoir" waters are in their most concentrated form.

If the "reservoir" waters were completely mixed and no other sources of variation other than the tide and the residual variance were present, then σ_o^2 would be equal to about 0.10 and would of course represent the combined experimental and theoretical variations. The difference $(\sigma_o^2 - \sigma_e^2)$, is the amount of variation due to incomplete mixing. The amount of mixing observed is rather uniform as can be deduced from the chi-square (χ^2) analysis of Table II, except for the sample obtained at 1440 which was characterized previously as being of a streaky nature.

The application of the method presented to other factors of stream pollution such as Biochemical Oxygen Demand, Dissolved Oxygen, water temperature, etc., for cycles of various periods is obvious. Other cyclical variables encountered in the sanitary engineering field such as the chlorine dosage required for a given chlorine residual may well be amenable to similar analyses, and the dosage can then be programmed accordingly.

Table I

MPN DATA

July 22, 1951

Explanation of Columns

Column (1) = Time of Observation, Eastern Daylight Time. Twenty-Four Hour Clock.
Column (2) = Time Elapsed from Last Previous Time of Maximum Ebb Current (Hours)
Column (3) = Time Interval Between Maximum Ebb Currents
Column (4) = $\frac{(2)}{(3)} \times 360^{o}$
Column (5) = MPN per 100 ml. = Y
Column (6) = sine (4)
Column (7) = cosine (4)
Column (8) = log (5)
Column (9) = K_3(6)
Column (10) = K_4(7)
Column (11) = Predicted Values of log MPN = K_1 + (9) + (10)
Column (12) = antilog (11)
Column (13) = (11) - (8)

TABLE IA Data MPN/ = July 22, 1951

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
0545	2.3667	12.3500	69.0	7,900	∠ .9336	∠.3584	3.8976	∠.60213	∠.02322	4.69168	49,170	∠.7941
0600	2.6167	12.3500	76.3	70,000	∠ .9716	∠.2368	4.8451	∠.62664	∠.01534	4.70831	51,090	-.1368
0620	2.9500	12.3500	86.0	23,000	∠ .9976	∠.0698	4.3617	∠.64341	∠.00452	4.71426	51,790	∠.3526
0640	3.2833	12.3500	95.7	23,000	∠ .9951	-.0993	4.3617	∠.64180	-.00643	4.70170	50,320	∠.3400
0700	3.6167	12.3500	105.4	23,000	∠ .9641	-.2656	4.3617	∠.62181	-.01721	4.67093	46,870	∠.3092
0720	3.9500	12.3500	115.1	22,000	∠ .9056	-.4242	4.3424	∠.58408	-.02748	4.62293	41,970	∠.2805
0740	4.2833	12.3500	124.9	13,000	∠ .8202	-.5622	4.1139	∠.52900	-.03642	4.55891	36,220	∠.4450
0800	4.6167	12.3500	134.6	33,000	∠ .7120	-.7022	4.5185	∠.45921	-.04549	4.48005	30,200	-.0384
0820	4.9500	12.3500	144.3	46,000	∠ .5835	-.8121	4.6628	∠.37633	-.05261	4.39005	24,550	-.2727
0840	5.2833	12.3500	154.0	49,000	∠ .4384	-.8988	4.6902	∠.28275	-.05822	4.29086	19,540	-.3993
0900	5.6167	12.3500	163.7	23,000	∠ .2807	-.9598	4.3617	∠.18104	-.06218	4.18519	15,320	-.1765
0920	5.9500	12.3500	173.4	13,000	∠ .1149	-.9934	4.1139	∠.07411	-.06435	4.07609	11,910	-.0378
0940	6.2833	12.3500	183.2	3,300	- .0558	-.9984	3.5185	-.03599	-.06468	3.96566	9,240	∠.4472
1000	6.6167	12.3500	192.9	64,000	- .2233	-.9748	4.8062	-.14402	-.06315	3.85916	7,230	-.9470
1020	6.9500	12.3500	202.6	4,900	- .3843	-.9232	3.6902	-.24786	-.05981	3.75866	5,737	∠.0685
1040	7.2833	12.3500	212.3	4,900	- .5344	-.8453	3.6902	-.34467	-.05476	3.66690	4,644	-.0233
1100	7.6167	12,3500	222.0	3,300	-.6691	-.7431	3.5185	-.43185	-.04814	3.58665	3,861	∠.0682
1120	7.9500	12.3500	231.7	3,300	- .7848	-.6198	3.5185	-.50616	-.04015	3.52002	3,311	∠.0015
1140	8.2833	12.3500	241.5	2,300	- .8788	-.4772	3.3617	-.56679	-.03091	3.46863	2,942	∠.1069
1200	8.6167	12.3500	251.2	2,300	- .9467	-.3223	3.3617	-.61058	-.02088	3.43487	2,722	∠.0732
1220	8.9500	12.3500	260.9	680	- .9874	-.1582	2.8325	-.63683	-.01025	3.41925	2,626	∠.5868
1240	9.2833	12.3500	270.6	680	-1.0000	∠.0105	2.8325	-.64496	∠.00068	3.42205	2,643	∠.5896
1300	9.6167	12.3500	280.3	3,300	- .9839	∠.1788	3.5185	-.63458	∠.01158	3.44333	2,775	-.0752
1320	9.9500	12.3500	290.0	7,900	- .9397	∠.3420	3.8976	-.60607	∠.02216	3.48242	3,037	-.4152
1340	10.2833	12.3500	299.8	7,900	- .8678	∠.4970	3.8976	-.55970	∠.03220	3.53883	3,458	-.3588
1400	10.6167	12.3500	309.5	22,000	- .7716	∠.6361	4.3424	-.49765	∠.04121	3.60989	4,073	-.7325
1420	10.9500	12.3500	319.2	13,000	- .6534	∠.7570	4.1139	-.42142	∠.04904	3.69395	4,943	-.4199
1440	11.2833	12.3500	328.9	350,000	- .5165	∠.8563	5.5441	-.33312	∠.05547	3.78868	6,147	-1.7554
1500	11.6167	12.3500	338.6	23,000	- .3649	∠.9311	4.3617	-.23535	∠.06032	3.89130	7,786	-.4704
1520	11.9500	12.3500	348.3	7,900	- .2028	∠.9792	3.8976	-.13080	∠.06343	3.99896	9,976	∠.1014
1540	12.2833	12.3500	358.1	4,900	- .0332	∠.9995	3.6902	-.02141	∠.06475	4.10967	12,870	∠.4195
1600	0.2667	12.4167	7.7	23,000	∠ .1340	∠.9910	4.3617	∠.08642	∠.06420	4.21695	16,480	-.1447
1620	0.6000	12.4167	17.4	7,900	∠ .2990	∠.9542	3.8976	∠.19284	∠.06181	4.32098	20,940	∠.4234
1640	0.9333	12.4167	27.1	33,000	∠ .4555	∠.8902	4.5185	∠.29378	∠.05767	4.41778	26,170	-.1007
1720	1.6000	12.4167	46.4	28,000	∠ .7242	∠.6896	4.4472	∠.46708	∠.04467	4.57808	37,850	∠.1309
1740	1.9333	12.4167	56.1	7,000	∠ .8300	∠.5578	3.8451	∠.53532	∠.03613	4.63778	43,430	∠.7927
1800	2.2667	12.4167	65.7	17,000	∠ .9114	∠.4115	4.2304	∠.58782	∠.02666	4.68081	47,950	∠.4504
1820	2.6000	12.4167	75.4	13,000	∠ .9677	∠.2521	4.1139	∠.62413	∠.01633	4.70679	50,910	∠.5929

TABLE II ANALYSIS OF RESIDUAL ERRORS

1	2	3	4	5	6	7	8	9
-1.99 / -1.70	-1.85	1	-2.01	-4.02	49.997	0.028	0.01	98.01
-1.69 / -1.40	-1.55	0	-1.71	-3.42	49.969	0.029	0.08	.08
-1.39 / -1.10	-1.25	1	-1.41	-2.82	49.760	1.081	0.41	.85
-1.09 / -0.80	-0.95	1	-1.11	-2.22	48.679	3.941	1.50	.17
-0.79 / -0.50	-0.65	2	-0.81	-1.62	44.738	10.124	3.85	.89
-0.49 / -0.20	-0.35	5	-0.51	-1.02	34.614	18.338	6.97	.56
			(-0.21	-0.42	16.276			
-0.19 / +0.10	-0.05	11	(+0.08	+0.16	6.366	22.632	8.60	.67
0.11 / 0.40	+0.25	7	+0.38	+0.76	27.637	21.281	8.09	.15
0.41 / 0.70	+0.55	8	+0.68	+1.36	41.309	13.672	5.20	1.51
0.71 / 1.00	+0.85	2	+0.98	+1.96	47.500	6.191	2.35	.05
1.01 / 1.30	+1.15	0	+1.28	+2.56	49.477	1.977	0.75	.75
1.31 / 1.60	+1.45	0	+1.58	+3.16	49.921	.444	0.17	.17
		38				99.918	37.98	103.86

Mean = 0.02289

σ^2 = 0.2498 σ = 0.4998

1. Class Range (Log MPN (predicted) - Log MPN (observed).
2. Mid Point of class range.
3. Number of observations.
4. Deviation of class limit from Mean.
5. (4) ÷ σ
6. Percent of area between class limit and mean.
7. Percent of area in class interval.
8. Theortical Frequency.
9. $\chi^2 = \frac{(\text{Observed} - \text{Expected})\ 2}{\text{expected}}$

TABLE III

MPN'S FOR 20 REPLICATES OF ONE SAMPLE

1.	790,000./ml.	11.	230,000./ml.
2.	1,300,000.	12.	460,000.
3.	330,000.	13	790,000.
4.	2,220,000.	14.	1,300,000.
5.	330,000.	15.	490,000.
6.	1,300,000.	16.	330,000.
7.	490,000.	17.	490,000.
8.	1,110,000.	18.	1,700,000.
9.	460,000.	19.	1,700,000.
10.	790.000.	20.	230,000.

TABLE IV

ANALYSIS OF VARIANCE

(1)	(2)	(3)	(4)
Total Explained by curve $= K_3\Sigma Y\sin\frac{t}{p} + K_4\Sigma Y\cos\frac{t}{p}$ $-2K_1\left[K_3\Sigma\sin\frac{t}{p} + K_4\Sigma\cos\frac{t}{p}\right]$ $-2K_3K_4\Sigma\sin\frac{t}{p}\cos\frac{t}{p}$	2.167,000.	2	1.083,500
Residual (by difference)	9.129,896.	36	0.253,608
Total $=\Sigma Y^2 - \frac{(\Sigma Y)^2}{N}$	11.296,896	38	

$$F = \frac{1.083,500}{0.253,608} = 4.27$$

$$0.01 < P < 0.05$$

(1) Source of Variance
(2) Sums of Squares
(3) Degrees of Freedom
(4) Mean Squares

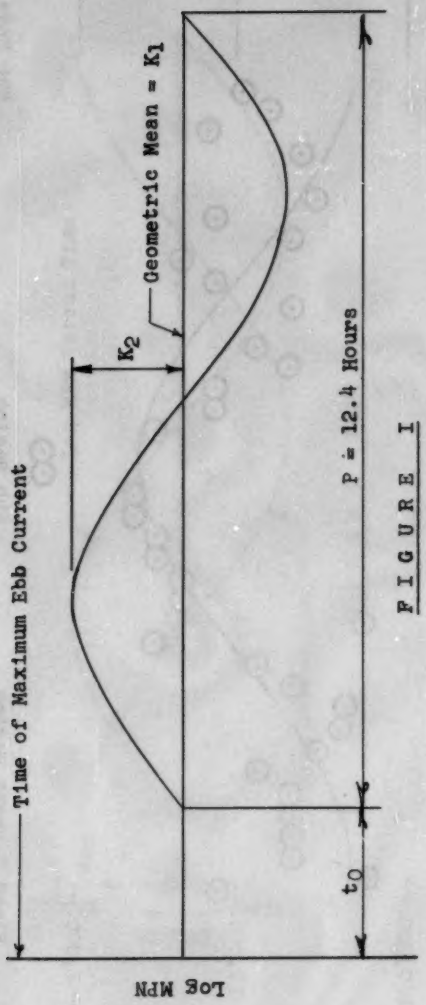
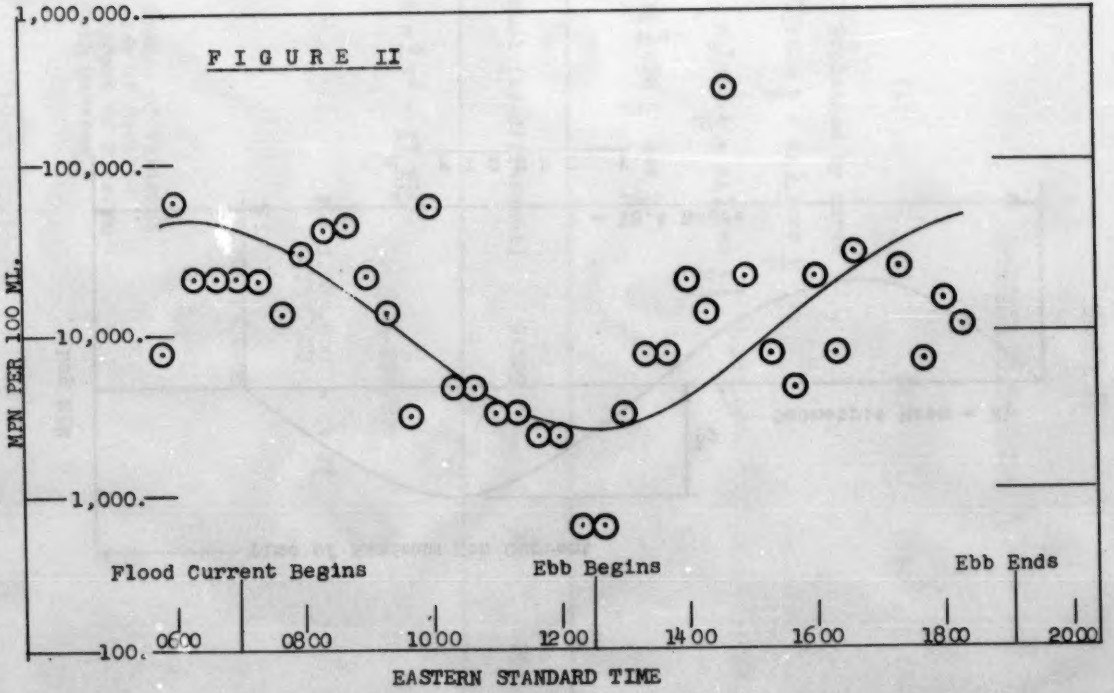

Log MPN
Time of Maximum Ebb Current
Geometric Mean = K_1
K_2
P = 12.4 Hours
t_0

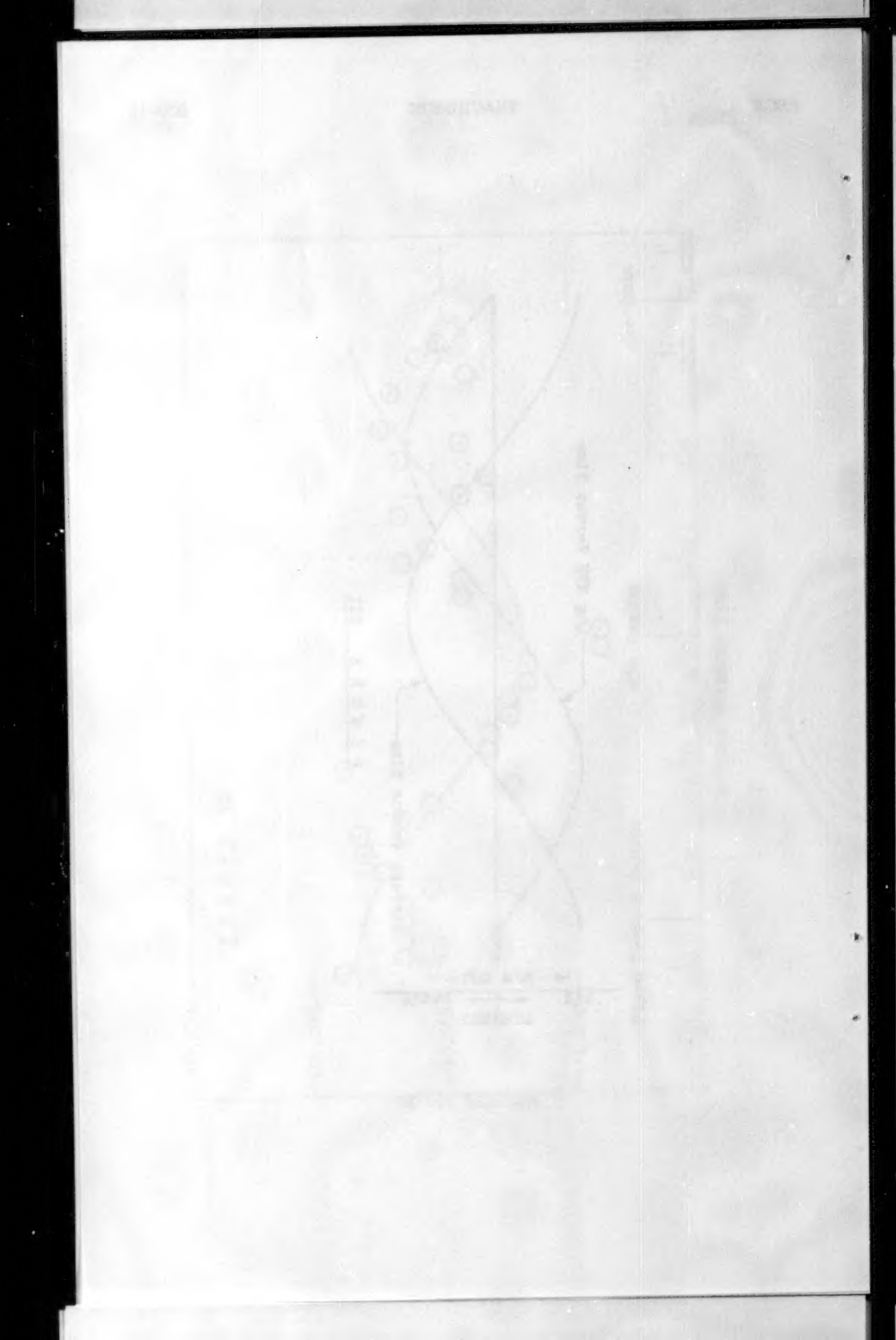
FIGURE I

FIGURE II


1,000,000.
100,000.
10,000.
1,000.
100.
MPN PER 100 ML.
Flood Current Begins
Ebb Begins
Ebb Ends
0600
0800
1000
1200
1400
1600
1800
2000
EASTERN STANDARD TIME

Log MPN Versus Time
Current Versus Time
LOG MPN
CURRENT
Flood
Ebb

FIGURE III



Paper 931 SA 2

JOURNAL

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WATER SUPPLY IN ARCTIC AREAS: DESIGN FEATURES*

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(Proc. Paper 931)

In his diary for 1932, Father Etienne Bazin, a Catholic Missionary in the Iglulik Islands located well above the Arctic Circle in Canada, made the following entries:

"July 30—The mosquitoes are plentiful this year. . . . . .something frightful; you can't put your nose out of the door. The ice is beginning to loosen; soon it will be gone."

"August 25—Yesterday we had a white whale hunt and harpooned three."

"August 27—There is already ice on the lakes. . .the first snow fell today. As you see, summer is not very long."

To those here in a temperate climate, Father Bazin's comment probably should have been recorded as the understatement of his day. But the Arctic abounds with the unbelievable and the parodoxical. In winter, the sun barely peeks over the horizon, if at all. Yet, many of the extended nights are wildly alight with the shimmering colors of the Aurora Borealis. In summer, the Fairbanks ball team traditionally starts one ball game each year at 12 o'clock midnight, without benefit of floodlights.

One may bask in the ninety degree weather of July but the frigid winds of January will freeze exposed flesh in one minute. Although most of the underground soil is permanently frozen, in some places to depths of more than 1,000 feet, numerous hot springs are found. The Great Siberian Plain of Asia and the Great Central Lowland of North America which comprise most of the Arctic land mass are less than 1,000 feet in elevation. The Arctic seas are relatively shallow. On the other hand, most of Greenland, which is the largest most northerly land and ice mass, lies more than a mile high.

Water supply is also part of the paradox of this land of contrast and extreme. The precipitation is less than 12 inches yearly, yet most of the Arctic and sub-Arctic is a veritable welter of ponds and lakes. Design of a satisfactory community water system poses more than the usual complications. This is not to say that the Arctic is impregnable, for man and his machines

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have assaulted the Arctic during the past decade on a scale not equalled during all previous Arctic History including the Alaska Gold Rush. This assault was sparked by the demands of war. It is being assisted by the discovery of the vast resources of the area and man's ability to procure and use them. The quest for precious and base metals, petroleum, iron and ferro-alloys in the sub-Arctic is becoming increasingly rewarding. The harvest of the tremendous coniferous forests of the sub-Arctic region is only now just beginning.

The climax of the assault, at least to date in the western hemisphere was probably reached in Operation Blue Jay at Thule, Greenland, less than 900 miles from the North Pole. Here in less than 100 days a modern air base was built complete with landing strip, hangars, and comfortable living accommodations for several thousand people. One of the principal problems to be overcome at this site is that of obtaining a suitable water supply.

The conquest of the Arctic appears to be well under way. Numerous water systems, both civilian and military, are completed and expanding. In Alaska alone, some 20 million dollars have been spent for this purpose. The Distant Early Warning Stations being stretched across the top of the world by the Armed Forces must also provide water systems, although probably on a small scale. According to a recent newspaper report, a city of 50,000 persons is being constructed in the Siberian Arctic by soviet prisoners of war.

The literature is gradually accumulating to relate the problems encountered and the means for combating them. Engineers concerned with design have increased in number and they are making rapid progress in learning and applying the disciplines imposed by the Arctic. Many designs which came into being have now had the benefit of observation so that an evaluation of their adequacy can at least be started.

Know the Arctic

It should be mentioned that this discussion refers mostly to the inland Arctic areas and those areas bounded by the Arctic Seas or underlain with permanently frozen ground (permafrost). Most of the communities in the north first settled along the sea coasts of the North Pacific and North Atlantic Oceans. Currents from these oceans and trade winds are responsible for average annual temperatures of 30° F. at locations well above the Arctic Circle. Much of the coast of Greenland enjoys these moderating effects, whereas the severe inland Arctic averages 20 to 30 degrees F. below the above figure.

The lower limit of the sub-Arctic is generally considered to extend as far south as approximately the 50th parallel which includes most of Canada on the North American continent; on the continent of Asia it reaches about to the 45th parallel, a few hundred miles north of Moscow.

The principal effect of the prevailing low temperatures and the permafrost manifests itself through the retardation of biological and chemical reactions and the changed physical properties of fluids, gases, and solids. Some of these effects are listed:

1) Ground water obtained from the permafrost area through wells is subject to freezing in the withdrawal pipe from underpumping, and freezing in the aquifer from over-pumping.

2) Frost heave may damage well pump installations. Freezing of pump bearings is an ever-present hazard.

3) Surface waters may freeze to the bottom whether they are in the form of a flowing stream or a lake.

4) The creation of an impoundment by construction of a dam may alter the underground thermal regime to such an extent as to endanger the stability of the structure.

5) Intakes in surface sources are readily subject to clogging from frazil ice.

6) All chemical treatment processes are slowed down to the extent that reaction chambers may become inadequate in capacity.

7) Subsidence of chemical floc and natural turbidities proceeds slowly.

8) Increased viscosity of water retards filtration.

9) Drainage devices such as provided for hydrants are rendered inoperative.

10) Foundations of structures placed on permafrost are subject to complete failure.

11) The distribution of water through piping systems may be fraught with endless difficulties by frost penetration and subsequent freezing. This is also true of the storage of water.

12) Condensation and freezing on the exterior walls of buildings add to the operator's woes and seriously deteriorates electrical and mechanical works mounted thereon.

In addition to the effects of low temperature and permafrost, there are conditions inherent in Arctic waters which must be taken into account. These are:

1) Surface waters possess fairly high pH values and frequently contain considerable glacial silt in suspension.

2) Ground waters are generally highly mineralized. Iron is usually present in objectionable quantities.

It is apparent, from the conditions just cited, that to design properly for Arctic waters, one must know Arctic conditions. It has been demonstrated too often that to ignore this fact is to invite disaster.

Operating Conditions

Design of a water system must also recognize the need for relative simplicity to insure operation during extreme cold. Whenever an operator must work outdoors during this period, his major concern is keeping warm. Remote locations of intakes and storage facilities will, therefore, suffer from inadequate inspection. Utilidors not easily entered complicate repair and replacement.

Operation of Arctic water systems in remote areas is complicated by the fact that trained personnel are scarce and oftentimes not capable of maintaining numerous automatic and intricate electrical and mechanical equipment. Moreover, distances are great and weather does not always accommodate the airplane, thus repair parts are difficult to obtain.

Economics and Design

It is well known that construction costs in this region are extremely high. In the vicinity of Fairbanks, Alaska they are at least four times the costs in Continental United States. The Alaska Public Works office reports bid prices

for 4-inch steel pipe in place at Palmer, Alaska at $7.42 per foot; 4-inch cast iron at $10.90 per foot. The costs for water systems for the Armed Services in even more remote areas have not been revealed. In view of such high costs, most Alaskan community utilities are assisted by funds from federal sources through the Alaska Public Works Agency. Even with this assistance, a city's indebtedness to purchase a suitable water system may run as high as $350 per capita.

Apparently there is no Federal relief to assist with operating costs, however, and the highly expensive plant is also expensive to operate and maintain. It has been estimated that the maximum cost at Fairbanks for power alone will be $36,000 per year for a present population of 7,500. Add to this the cost of labor, chemicals, and general maintenance, for a five million dollar installation, plus annual charges for interest and retirement of the city's share of the indebtedness for the first cost; the estimated annual total cost per capita is almost as much as the initial outlay, or somewhere between $250 and $300. Thus, in spite of Federal beneficence, water systems in the Far North cost a great amount of money. This is a further restriction on the designer.

Design Features

The following features of design have been compiled from numerous sources. They are by no means all-inclusive. They have resulted largely from experience and from the research of individuals and various agencies. Some are suggestions to remedy deficiencies which have been observed in existing systems. They are presented with the sole intention of aiding those concerned with the creations of new systems or improving existing works.

Basic Factors

Design should probably first consider the temperature and extent of ground frost in the soil for the area concerned. The factors which determine temperature and ground frost fall into two main categories, namely:

a) Factors due to geographical position
b) Factors due to local conditions

Those factors due to position are:

1) Latitude
2) Altitude
3) Proximity to bodies of water
4) Terrain
5) Exposure to the sun

Those factors due to conditions are:

1) Ground cover
2) Moisture content of soil
3) Soil density
4) Soil color
5) Soluble salts in soil moisture
6) Thermal properties of soil

Familiarity with these factors should automatically bring to light the special conditions which will govern design in a given locality.

A. Selection of a Water Supply Source

1) Surface Waters

In comparatively rough terrain such as in Alaska, surface water runoff is rapid in all areas except lowlands near rivers and river deltas. In permafrost areas, water sheds cannot normally be expected to retain precipitation similar to those of the temperate climates, since percolation obviously is restricted. Minimum stream flows exist during late winter and early spring.

Impoundment of surface waters must be preceeded by a thorough study of underground conditions. The stored water may thaw permafrost, which, depending on the nature of the soil in the permafrost, may in turn open underground channels for the release of the stored water. Thawing of permafrost under dam structures may render the foundations unstable, especially where soil particles are finely divided and possess a high moisture content and where no other precautions are taken.

Intakes should be placed in a suitable depth of water to allow ice cover to protect against frazil ice. This is not necessarily the sole solution to this problem, however. Control of velocity of flow may be indicated as well supplying auxiliary heat.

2) Ground Waters

Ground waters are usually warmer than surface waters in winter. Other factors being equal, this higher temperature means less auxiliary heat being added to the distribution system. The deeper ground waters are usually the warmest.

Water production from wells has been found to fluctuate, reaching a minimum in late winter and early spring. Some well supplies have become entirely depleted. Test pumping during this critical period is, therefore, advisable to determine reliable yields, prior to final development of the well.

The following points of natural topography are indicative of the possible availability of ground water.

a) Wide, shallow exposed channels, glacial streams, or alluvial fans.
b) Slopes at, or near, the foot of a mountain or hill where movement of ground water down the slope may be intercepted.
c) Locations immediately downgrade from poorly-drained areas such a as muskeg swamps, which, due to retention by heavy vegetation are apt to yield a continuing supply during winter.
d) Any sudden reduction in natural channel gradient or channel width in such a way that the constriction would tend to accumulate water.

Where water sources are difficult to find on the ground, aerial studies have been of help. Color photography from the air during the late summer season may show up areas of greener and denser vegetation which are indicative of the presence of ground water.

3) General

The sites selected for a water source should minimize the lengths of transmission lines as this saves heat and reduces inspection and upkeep. If possible ground water sources may be located near a source of waste heat as in the case of Fairbanks, Alaska, near the City Power Plant. In an attempt to obtain a water supply in winter at something more than

freezing temperature, engineers at Anchorage, Alaska, have considered the underground storage of summer flows from a nearby river. Recharging of ground water supplies in Sweden has been practiced for many years.

B. Well and Pumping Facilities

In order to avoid damage from seasonal frost and from permafrost, the well and pumping works design must incorporate some features in addition to the orthodox weather-proofing practice. Concrete slabs are poor insulators and they transmit the heat from water in well casings, intakes, and discharge lines to the outside cold sufficient to cause the water to freeze. Concrete slabs may also occasionally thaw the underlying permafrost. When this moisture re-freezes heaving takes place which may damage pumping facilities and otherwise throw vertical well pumps and casings out of alignment.

Freezing of well casings in contact with permafrost is prevented by a judicious program of pumping. Where pumping costs are excessive or pumping is otherwise not advisable, some other means must be provided to prevent freezing. Resistance-heating cables and hot water have been employed with some success.

In pumping wells, care also should be taken not to exceed recoverable yield of the aquifer as this may deplete the total quantity of water movement to the extent that freezing may occur from the encroachment of surrounding permafrost. Where they are feasible, observation wells in addition to the water well, are obviously desirable; first, in determining water yields, and secondly, in charting the influence of various rates of pumping.

Temperatures of permafrost should be determined for water wells and whenever permafrost is encountered in construction. Temperatures are known to vary from 15^{o} F. to 32^{o} F. Permafrost near 32^{o} F. may be removed with reasonable assurance that it will not return. Low temperature permafrost will probably freeze back after a strata has been disrupted.

C. Water Treatment

Ground waters will usually require iron removal. High iron content appears to be characteristic of most ground waters from Alaska to Greenland. The removal of hardness may also be desirable.

Treatment required for surface waters varies considerably. In Alaskan areas waters derived from melting glaciers carry large quantities of glacial silt thus necessitating coagulation and filtration. Other Alaskan rivers and rivers in northern Canada are turbid and colored during spring and summer run-off and treatment should be employed. A fairly high pH is common in these waters; this factor combined with low temperatures obviously retards disinfection by chlorination. The large lakes in northern Canada are usually suitable without treatment except chlorination. Waters from muskeg areas are typically colored and soft with low pH values.

Use of smaller lakes in the very cold regions should be approached with caution. In one instance where this is done the water is quite satisfactory during the first part of the winter. But as the cold weather continues the ice cover gets thicker and water quantity gets less with the result that organic matter and minerals become highly concentrated. In 1949 from January 15 to February 2, the total dissolved solids of this supply were doubled. Whereas one could draw normal, relatively clear water in

December from this system, the liquid emerging from the tap as winter wore on literally resembled dilute sewage both in appearance and odor.

Treatment processes must take into account that,

a) The increased viscosity of low temperature water permits the carrying of greater silt loads; requires settling chambers and filter area nearly double that of warmer waters.
b) The slowing-up of chemical reactions so that flocculation basins may have to be enlarged up to three times the orthodox design.
c) The increased solubility of oxygen which may aggravate corrosion problems when the water is warmed.

The above implications point out the desirability of raising the temperature of water prior to treatment. This has been done at the new treatment plant at Fairbanks, Alaska, where waste heat from the power plant is utilized. Even though waste heat may not be available, it behooves the designer to balance the costs of deliberately furnishing auxiliary heat against the increased costs of treating low-temperature water.

In addition, experimentation in chemical dosage, flocculation and settling over the wide range of conditions encountered, are doubly important. Laboratory facilities obviously should be included in any treatment plant design.

Adequate insulation for outside walls of treatment plant construction, and positive means for humidity control should be provided to prevent condensation and freezing of moisture. Treatment plant operation would probably be facilitated by the use of electrical or air control of valves and rate-of-flow controllers as well as indicating mechanisms, rather than by hydraulic methods.

Where seasonal turbidities are the only objectional constituent in a small water supply, consideration should probably be given to the use of diatomaceous-earth filters. The initial cost of this system is only a fraction of the cost for the orthodox coagulation and filtration processes and operation is greatly simplified. Presettling should be employed in this method, however, when turbidities are high. One small community in Canada has used these filters with apparent success.

Distribution and Storage

The satisfactory distribution of water through piped systems is probably the most vexing problem facing the designer. Many different types of systems are in use—most of them have had troubles of one kind or another. Where heat is distributed from central heating plants, such as at military bases, the prevention of freezing is somewhat simplified. The usual practice in this case is to construct large conduits called utilidors, which contain both the heat and the water transmission mains.

Other utilities including sewers may also be placed in the utilidor. This practice is not without its objections, however, because sewer lines jeopardize the safety of the water mains. Experience has shown that the large conduit may settle out of alignment causing water and sewer lines to become disjointed or broken. Water lines have been observed completely inundated with sewage. Where water and sewage lines are placed in a single utilidor, the conduit should be of the walk-through type. If finances do not permit this costly construction, it is suggested that a smaller utilidor be sloped to drain throughout, and that the sloped lines terminate in buildings or stations which

require operating personnel to enter at least daily and thus observe conditions. The roof of the utilidor, except perhaps for the walk-through type, should be so constructed that access to the interior is readily accomplished. Removable slabs with lifting hooks have been satisfactory from this standpoint.

Distribution of water without the use of utilidors is commonly practiced, but auxiliary heat must usually be provided. This heat may be added at the source or at strategic points in the system. The source of heat is optional and probably depends on the local situation as to availability. Temperature-sensing instruments are essential for control and the present-day devices will insure almost automatic operation, if this is desired, to keep the water above freezing. Heat is conserved by maintaining a temperature only slightly above 32°F. The cost of insulating water pipe should be balanced against the cost of increased heating. Materials locally available such as peat, moss, sands, and clay have been used to economic advantage.

Some insulation must surround water mains placed in permafrost to avoid excessive thawing and subsequent settling. This contingency can usually be met, however, by backfilling with suitable materials such as sand and gravel if it is available, to form a more substantial bedding for the pipe.

The recirculation of water through the system appears to be favored by many. Although the two-pipe system with a smaller-sized return line has been used, the single-pipe system is probably to be preferred. This latter method assumes that service connections can be made without danger of freezing. Here, again, auxiliary heat may be used although a recirculating service line is entirely feasible and is recommended.

To avoid seasonal frost water mains should be laid below the average depths to which the frost penetrates. A ten-foot laying depth has been used in systems now operating although where good insulation and heating are provided much shallower trenches have been used. One system places most of the water mains in insulated boxes on the ground surface. Another system provides a cover of only 5 to 6 feet.

Storage

Storage aboveground has followed the practice in many of our Northern States where riser pipes are well-insulated and auxiliary heat is introduced at the base of the risers. Recirculation of water in the large-diameter risers may be sufficient, but temperature indicators should be provided in this case. In addition, the elevated tanks themselves should be enclosed with an insulated outside covering for most economical operation. The top section of the 12-story city hall in Kemi, Finland, houses the city water storage tank.

Storage in surface and subsurface tanks and reservoirs should consider the underground conditions, the nature of any permafrost and the soil contained in it. The relatively poor insulating qualities of concrete should be kept in mind, and supplemental insulation should be provided where transfer of heat must be retarded.

Structure Foundations

Various attempts have been made to provide stable foundations for structures placed in or above permafrost such as:

a) Removal of finely divided soils with high-moisture content and replacement with more stable material such as gravel or crushed rock.
b) Jetting and driving piling into the permafrost and allowing them to freeze in place.
c) Insertion of refrigerating coils into the underground soil to maintain continuous freezing.
d) Use of plenum chambers below structures through which the outside air is circulated.
e) Construction completely above the surface of the ground so that the outside air may circulate freely.

Where plenum chambers are installed for circulating cold outside air during winter, consideration might be given to the insertion of a refrigerating unit in the air circulating ducts for summer-time cooling. The employment of refrigeration coils to keep the ground frozen should be approached with caution. If the thermal regime is such that the permafrost is gradually thawing the freezing coils may eventually become completely inadequate regardless of their capacity.

SUMMARY

The design of water supplies to meet arctic conditions must take into account the rather severe restrictions imposed by the long, sub-zero winters and the permanently frozen ground.

To overcome these restrictions the designer should:

1) Thoroughly study local conditions of temperature, permafrost, and the availability of insulating materials.
2) Make allowance for the retardation of biological and chemical reactions.
3) Consider the personnel who will operate the system.
4) Provide for ease of operation, inspection, and maintenance.

In addition, the extremely high costs of labor, materials, and equipment, should be borne in mind so that the resulting utility will be within the owner's financial ability to purchase and, thereafter, to operate.

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Discussion of "SAND FILTRATION STUDIED WITH RADIOTRACERS"

by Donald R. Stanley
(Proc. Paper 592)

DONALD R. STANLEY,[1] M. ASCE.—As Mr. Camp has pointed out, some floc always penetrates completely through the filters. Therefore, strictly speaking, the concept of "penetration" of floc is unreal. This concept was used in the study conducted by a committee of the A.S.C.E. (1) and the writer found it useful for interpreting his experimental data. The important thing is to carefully define what is meant by the term.

Six methods of defining penetration were considered, these were:-

1. Visual observation of the depth to which floc passes into the sand. This method requires the least amount of work, but does not yield sharp results when penetration is uneven. It is probably more satisfactory with Fe than with other coagulants because of the high contrast in colour between Fe floc and the white sand.

   Fig. 14 shows six filters after a run in which both rates of flow and sizes of sand were varied. Except for filter No. 4 a very definite change in colour occurs at a certain depth in each filter. Theoretically, knowing that some floc always penetrates the full depth of the filter, it is probably incorrect to say that these depths at which the colour changes represent the depth of penetration. A very important observation during the experimental work was that the quality of the effluent deteriorated rapidly at about the same time that the defined "penetration" reached the bottom of the filter. In the case of filter No. 4 the defined penetration had reached the bottom of the filter before the photograph was taken.

2. Visual observation modified by a least squares fit of a straight line to plotting points on either side of the visual penetration point.

3. Filter depth within which a certain percentage of the floc has been removed.

4. Point on Fe distribution curve at which the concentration of Fe is essentially zero.

5. Fitting an appropriate function to the curve of Fe concentration against depth in the filter and calculating the desired percentage of total Fe removed. The depth of the filter corresponding to this percentile is defined as penetration. As an example, Fig. 15 shows the same data used for Fig. No. 3 plotted on semi-logarithmic paper. The equation that fits these three curves is as follows:-

$$y = c\, e^{-m x}$$

1. Pres., D. R. Stanley & Associates, Ltd., Edmonton, Alta., Canada.

where y = concentration of iron in mg/cc

x = depth in inches below surface

c = the intercept on the y axis at zero depth

m = the slope of the best fitting straight line on semi-logarithmic paper.

If the curve is treated as a frequency distribution curve with y representing relative frequency and the total area under the curve equal to unity then

$$1 = \int_0^\infty y\,dx = c \int_0^\infty e^{-mx}\,dx$$

$$= -\frac{c}{m}\, e^{-mx}\Big]_0^{-\infty}$$

$$\therefore\ 1 = \frac{c}{m}$$

$$\therefore\ c = m \text{ and } y = m\,e^{-mx}$$

With reference to the semi-logarithmic plot (Fig. 15) the values of m are as follows:-

Curve No. 1 $m = 0.496$

Curve No. 2 $m = 0.337$

Curve No. 3 $m = 0.253$

To obtain the 95 percentile for curve No. 1.

$$0.95 = 0.496 \int_0^{x95} e^{-0.496x}$$

$$= -\frac{0.496}{0.496}\, e^{-0.496x}\Big]_0^{x95}$$

$$x95 = 6.05 \text{ in.}$$

Similarly $x_{97} = 7.10$ inches.

x_{95}, x_{97} and x_{99} were computed for each curve and in each case x_{97} was closest to the observed penetration.

Another method of defining penetration using the semi-logarithmic plots as shown in Fig. 15 is to define it as the depth below which the best fitting straight lines show iron in lower than certain concentrations. For example, in Fig. 15 for a concentration of 0.1 mg/cc the defined penetrations would be 6.4, 10.4 and 14.4 inches for curves 1, 2 and 3 respectively. These figures agree closely with the observed penetrations.

6. Intersection of the curve of iron concentration against depth in filter with a straight line drawn parallel to but above the zero concentration line. The distance above the zero concentration line was chosen as one standard deviation of the background count. This was the method finally adopted.

In the final analysis it is desired to have the minimum depth of filter that will assure the required quality of effluent throughout a filter run. The quality of the effluent remained satisfactory until the defined penetration reached the bottom of the filter. The defined point of penetration then could have, alternatively and possibly more correctly, been called the point in the filter below which the concentration of suspended material is below the maximum allowable in the filter effluent.

Contrary to Mr. Camp's contention, the writer feels that the empirical equations relating the penetration index to the sand size and rate of flow have a very definite use in practice provided that one realizes the limited conditions under which the data were obtained. The formulation and graph are only important as to their form. The significant conclusion is simply that where various rates of filtration and sizes of sand are being considered in the design of filters the effect of these factors on the depth of bed should also be given consideration.

It was not intended that these formulations be applied directly to practice. Experiments were conducted only with beds of uniform sand using one type of floc produced under rigidly controlled conditions. Obviously, it would be necessary to investigate the effects of many more variables before the formulations can be applied quantitatively.

The lack of a time rate of head loss is not a valid reason against the use of the formulations in practice. In practice the time for backwashing is generally determined by the head loss through the filter which varies directly with the total floc passed into the sand and hence directly with time. At constant rates of flow and concentrations of suspended matter these two parameters are interchangeable but the loading parameter is more useful.

The writer does not agree with Mr. Camp's statement that it is not important how a particle reaches the surface of a sand grain but only that it must reach the surface. This statement is akin to saying that it is important to observe certain phenomena but not to understand the mechanisms by which they occur.

It appears that Mr. Camp did not carefully read the descriptions of experiments and conclusions outlined in the first two sections under "Special Tests." He states that the passing of distilled water through a filter without dislodging deposited floc does not indicate that when floc is passing into the bed the deposit of it may not be accompanied by the dislodging of another particle of floc just downstream therefore. The conclusions to which the above statement refers were drawn from an experiment in which non-radioactive floc was passed into a filter after first operating with radioactive floc and not on the experiment with distilled water as Mr. Camp infers.

As Mr. Camp has pointed out, the volume of floc particles is more important than their weight because it is the space which they occupy that restricts the flow. No experimental work was conducted to evaluate the effect of floc density. However, using observed head losses and concentrations, "specific volumes" of floc were evaluated (in mg/cc) by the use of a formula relating head loss with porosity. The determination of "specific volume" by this method was too inaccurate for application. The development of a reliable technique for determining such a parameter would be valuable in describing floc quality.

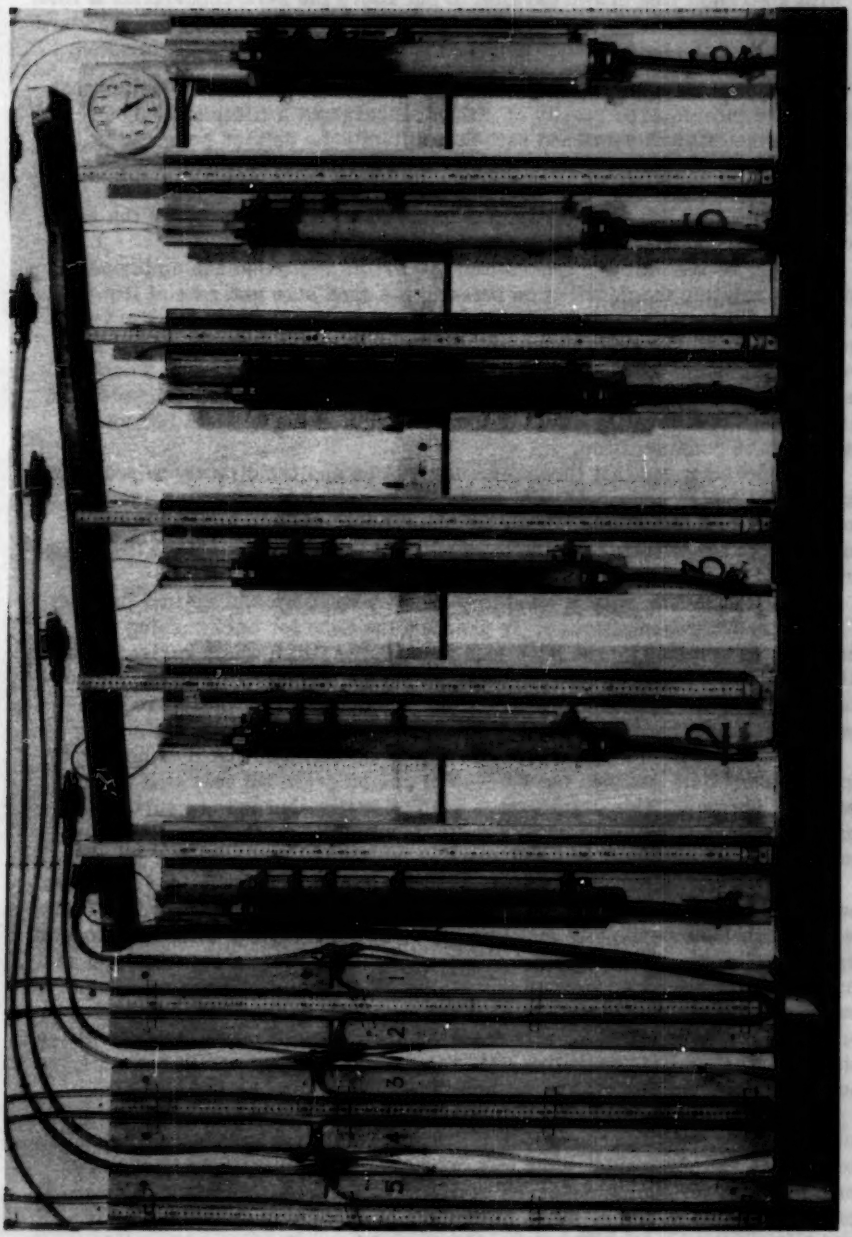


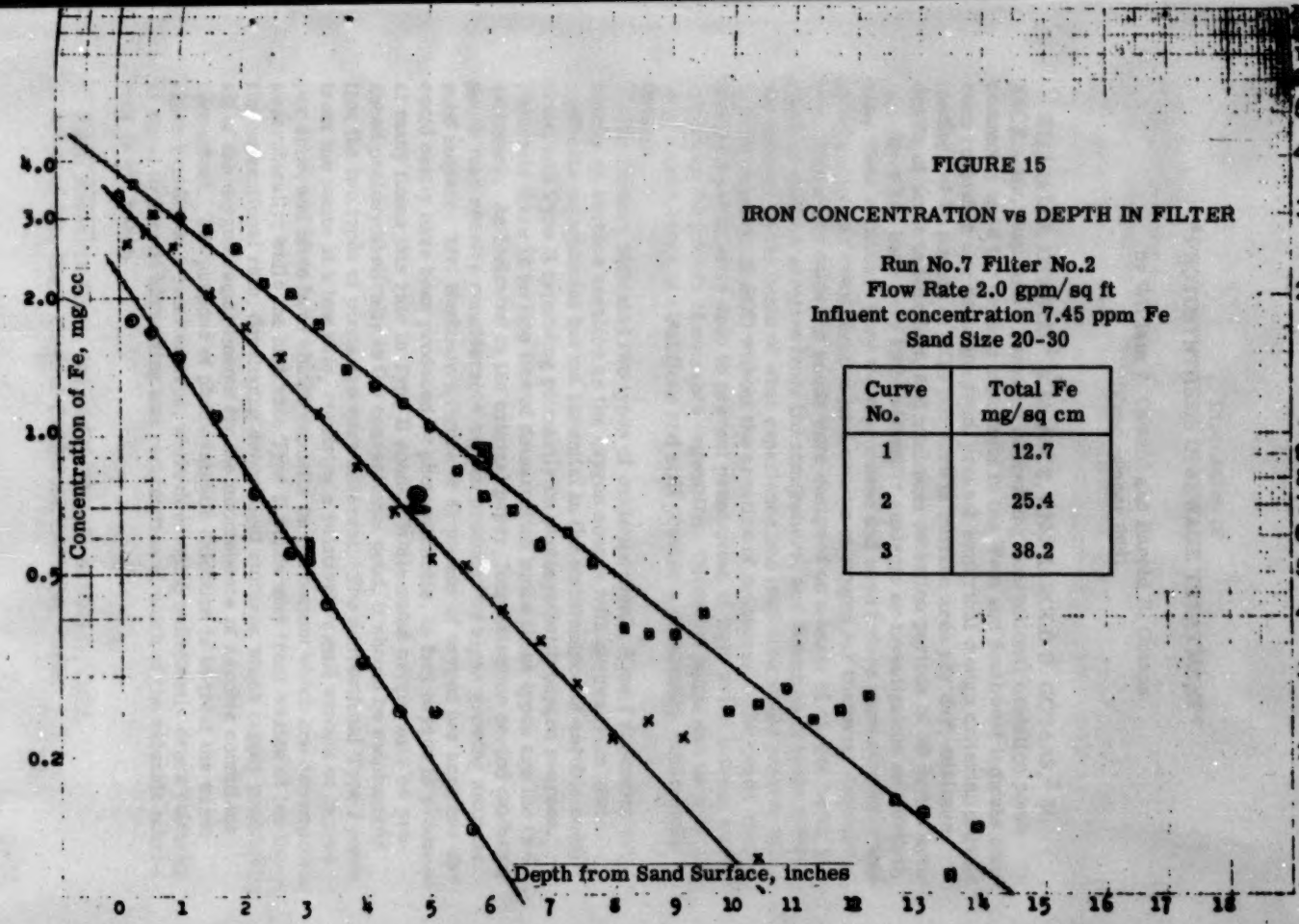
FIGURE 14
FILTERS & BLOWER MANOMETERS ON BLACKBOARD

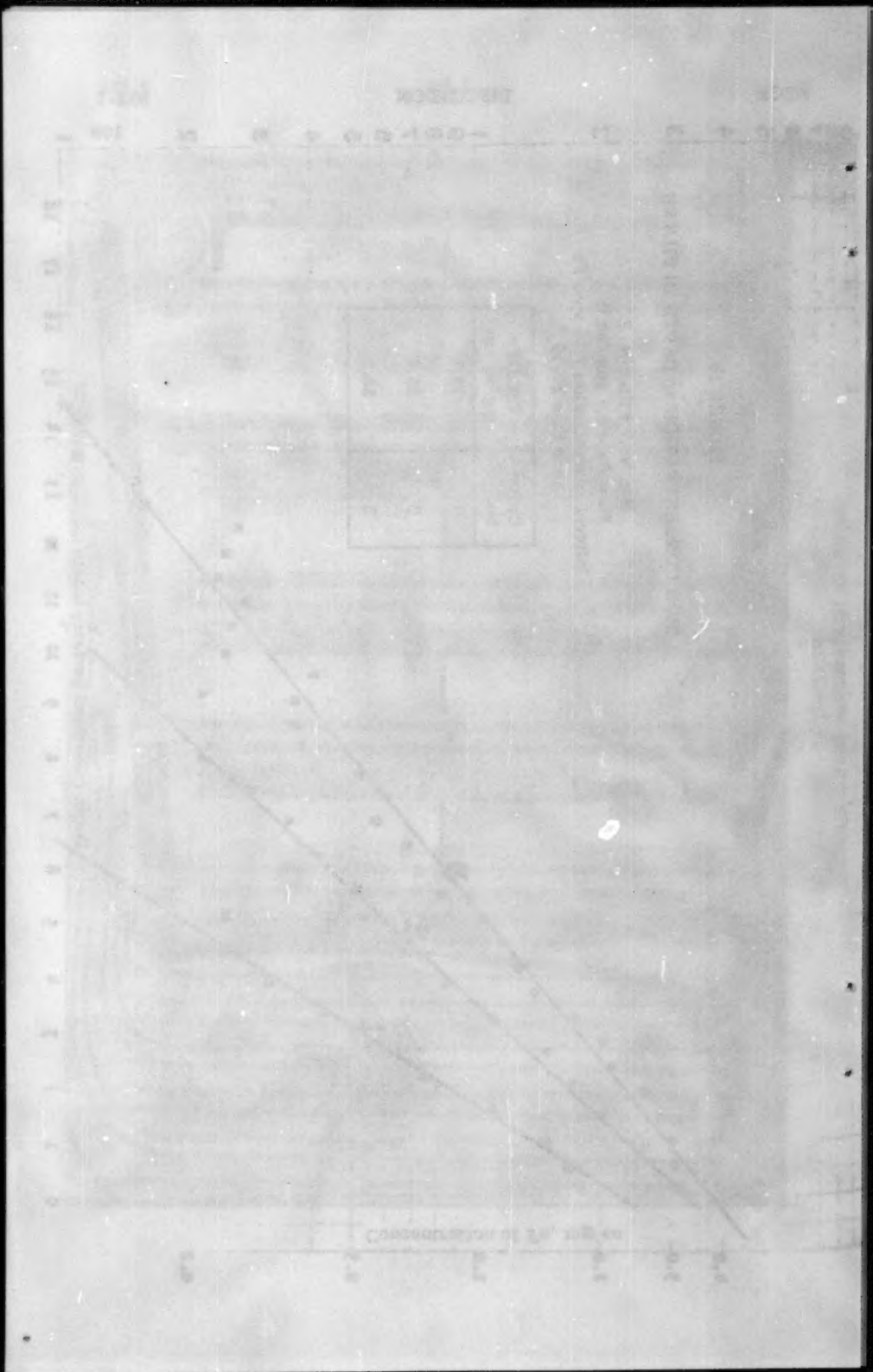
FIGURE 15

IRON CONCENTRATION vs DEPTH IN FILTER

Run No.7 Filter No.2
Flow Rate 2.0 gpm/sq ft
Influent concentration 7.45 ppm Fe
Sand Size 20-30

Curve No.	Total Fe mg/sq cm
1	12.7
2	25.4
3	38.2





Discussion of "PHOTOSYNTHESIS IN SEWAGE TREATMENT"

by William J. Oswald and Harold B. Gotaas
(Proc. Paper 686)

WILLIAM J. OSWALD,[1] A.M. ASCE, AND HAROLD B. GOTAAS,[2] M. ASCE.—Mr. Nusbaum's remarks concerning conventional oxidation ponds commonly used for sewage treatment in the West and Southwest indicate concern regarding any deviation from present empirical design criteria; organic loadings of 50 pounds of BOD per acre of surface area per day, minimum depths of three to four feet, and minimum detention periods of 30 days. Actually there has been very little scientific analysis or investigation on which to base these criteria. They were proposed and used before photosynthesis was given particular consideration as a source of oxygen for bacterial decomposition. The early oxidation ponds were designed on a basis of oxygen being supplied by surface aeration from the atmosphere, and detention periods were determined on the basis of what experience had indicated would provide satisfactory removal of BOD without the creation of nuisances. The ponds were designed sufficiently deep to prevent penetration of light to the bottom, thus inhibiting the growth of nuisance vegetation. Oxidation ponds can be operated over a wide range of conditions and still provide a reasonably satisfactory effluent.

The authors indicated two types of oxidation ponds, Type I depending primarily on surface aeration as the oxygen source with oxygen from photosynthesis coincidental but not essential in the maintenance of aerobic conditions; and Type II depending primarily on photosynthetic oxygen sources. Naturally there is no fine line of demarcation since these types are the two extremes. As indicated in the original paper, long detention period oxidation ponds may receive considerable oxygen produced by algae growing near the pond surface. Mr. Nusbaum's figure of 47 pounds of oxygen per acre per day could easily have been produced by photosynthesis, in fact oxygen is produced at many times this rate in Type II ponds. While some oxygen may be produced photosynthetically in the conventional pond, it should be emphasized that the two types of ponds are very different. The conventional Type I ponds treat the waste at a low rate, requiring a relatively small amount of oxygen per acre, and often have sludge deposits on the bottom which are decomposing anaerobically; while the high rate Type II ponds may treat wastes at ten times the conventional rate, developing dense algal cultures which supply practically all of the oxygen requirements for the maintenance of aerobic conditions throughout. The purpose of photosyntehtic oxidation is to treat the waste more rapidly on a smaller area, while developing sufficiently dense cultures of algae to permit harvesting and reclamation of much of the valuable nutrients in the waste.

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Conventional oxidation ponds designed for a BOD loading of 50 pounds per acre per day and a 30-day detention period often contain a sludge deposit on the bottom which undergoes anaerobic decomposition. Much of the dissolved and colloidal organic matter of the sewage is flocculated and settles to the bottom. If aerobic conditions are maintained in the upper part of the pond water, only the BOD of the non-settling material and the residual BOD of the anaerobically decomposed sludge is satisfied aerobically. Thus less oxygen is required than if all the organic matter is decomposed anaerobically. If the anaerobic sludge deposits are not too large, odor and nuisance conditions do not develop, since adsorbed atmospheric oxygen and photosynthetic oxygen in the supernatant may be sufficient to prevent odor nuisances. It may be seen that the average BOD loading per acre per day is a very inaccurate parameter, since the amount of deposition and anaerobic decomposition of BOD may be greater than the aerobic oxidation and the algal oxygen production may vary widely depending upon sunlight and other factors.

Oxidation ponds may be operated with detention periods and loadings between those of conventional Type I practice and values shown for the Type II, high rate ponds. The Type II pond requires some vertical mixing to keep the sludge, the bacteria, and the algae, in close proximity, so that the sludge will be aerobic and the algae can obtain the carbon dioxide and ammonia released by the bacteria. The authors have neither been able to develop dense algal cultures in deep, long detention period ponds, nor to find any conventionally designed ponds which produce high algal concentrations. There is a valid physical reason for this which was presented as equation 12 in the original paper. This equation shows that algal concentrations above 40 ppm can not be sustained in ponds 3 feet deep. Algal densities greater than 25 ppm are rare in the conventional pond, compared to 200 to 350 ppm in the Type II pond when ample sunlight and nutrients are available.

Studies indicate that the Type II, high rate ponds will have greatest economic possibilities when high quality secondary treatment is desired and the algae can be harvested, since chemical coagulation and settling of the algae produce an effluent water from which 95 to 97 percent of the BOD has been removed.

Mr. Nusbaum refers to the desirability of recirculation for overcoming shock loads on the pond. The operation of the high rate pond requires recirculation for seeding the influent waste with algae and adding oxygen to provide aerobic conditions initially. However, recirculation is desirable in the operation of any oxidation pond. In the Type II pond the amount of recirculation should be controlled since large recirculation hastens bioflocculation and settling of organic matter, thus retarding algal growth by separating the bacteria and nutrients from the algae which do not settle.

Reference was made to the die-away or removal of coliform bacteria in high rate oxidation ponds. If the algae are chemically coagulated and precipitated, practically all of the coliform organisms are removed to the sludge and the effluent may be satisfactorily filtered through a rapid sand filter. Many observations of the change in coliform densities in the high rate oxidation ponds have shown that the most probable number decreases from 10^7 to 10^8 in the sewage to 10^2 to 10^4 in the pond effluent. The variation between 10^2 and 10^4 is affected by the detention period and the depth. Ponds of shallow depths and longer detention periods in intense sunlight show lower coliform counts in the effluent.

Mr. Nusbaum indicated that oxidation ponds have low chlorine requirements

of around 2 to 3 ppm. The authors have found that the effluent from Type II ponds have a high chlorine demand due to their high algal concentration. They have also found that the effluent from Type II ponds after harvesting the algae, has a very low demand. This indicates, therefore, that the pond effluents referred to by Mr. Nusbaum contained very few algae, and therefore obtained little oxygen from photosynthesis.

The authors are unable to answer completely the question of why blue-green algae develop in the long detention period ponds and apparently do not develop in the high rate ponds. There are several factors which may influence the development of the blue-green algae. First, blue-green algae appear to grow better in partially nitrified sewage than do green algae, but are unable to compete with green algae when ammonia-nitrogen rather than nitrate-nitrogen is the nitrogen source. Only ammonia-nitrogen is available in the short detention period ponds. Secondly, blue-green algae will grow when sewage nitrogen is very low or absent since they have the capacity to fix nitrogen from the air. There is usually an excess of nitrogen in high rate ponds. The blue-green algae appear to grow around mats of organic sludge which may rise from the bottom of the pond where a sludge blanket has developed, indicating that the blue-green algae may require longer detention periods than green algae. It is therefore believed that the short detention periods of the Type II pond may exclude the growth of blue-green algae and permit green algae free access to the available nutrients.

Mr. Nusbaum has raised the question of growing Chlorella and other algae in ponds high in sodium chloride or total solids. There is little information indicating that brackish waters influence algal growth when total salt concentrations are less than 3000 ppm and no toxic minerals are present. It is possible that the noted unsuccessful results of attempts to seed Chlorella into brackish ponds were due to insufficient seed or excessively long detention periods. Since Chlorella is a small, rapidly growing alga, it dominates when detention periods are short and other larger, more slowly growing organisms, do not have the opportunity to overgrow it. Chlorella, Scenedesmus, and other of small, rapidly growing cells do not appear to predominate when the detention period is long.

Much additional study of photosynthetic oxygenation of sewage and separation of the algae from the liquid has been accomplished since the paper was written. Pilot plant investigations have shown that the algae and other suspended and colloidal matter can be continuously precipitated and an effluent produced which contains a very low BOD and bacteria count. The investigations indicate that during summer weather well over one ton, dry weight, of algae can be produced per million gallons of average domestic sewage. Based on pilot plant studies, the cost of growing, separating, and drying, the algae while also providing a high quality effluent is believed to be less than $100 per million gallons for average conditions. The algae contain from 45 to 55 percent protein as well as carbohydrates, fats, and minerals, and have an indicated value of well over $100 per ton as an animal food. Hence even though this process is now only in the developmental stage and accurate cost and operational data for a full scale plant are not available, it appears probable that Type II short detention period ponds and algae harvesting may provide high quality sewage treatment for a very low net cost. The most economical use of the process will be for cities requiring complete, year around treatment in areas where the climate and sunlight are satisfactory. This would generally be within 30^{o} to 35^{o} latitude from the equator. The process may

also be economical for use during the summer in places farther from the equator where only partial treatment is necessary during the cold winter period.

A rational formulation of the major fundamental biological, physical and chemical factors which govern production of oxygen and algae in sewage ponds together with engineering applicators of these fundamentals was presented. These principles are applicable under a wide variety of conditions and can be utilized in the design of either conventional or high rate type ponds whenever photosynthesis is a source of oxygen. Naturally, caution should be exercised in designing high rate ponds of large magnitude until the process has been operated at plant scale installations. Present information indicates, however, that conventional detention periods and loadings can be changed for greater efficiency and economy. Future design of oxidation ponds should include provision for conversion of conventional ponds to high rate operation, which is essential for the development of sufficient algae to permit economical harvesting and reclamation.

This process indicates the possibilities for much greater reclamation of the valuable nutrients now being wasted in costly treatment processes. The high rate process, which utilizes sunlight as a source of energy for growing algae to treat wastes and to reclaim nutrients, has one less degree of freedom for general application than the more conventional processes because of light requirements. However, in many localities sunlight is an unusually abundant item, and we can hardly afford to waste either nutrient materials or sunlight if ways can be developed for their economical utilization. Engineered control of photosynthesis offers a means to this end. Certainly civilization would have been retarded if engineers had chosen to allow natural processes to proceed only at their primordial rate.

Discussion of
"SEDIMENTATION IN RECTANGULAR BASINS"

by Claes N. H. Fischerström
(Proc. Paper 687)

FISCHERSTRÖM, CLAES N. H.,[1] M. ASCE.—The discussion by professor CAMP is extremely valuable and interesting. Some conclusions and also opinions expressed in letters received, seem however to indicate a certain misunderstanding of some in the writer's opinion important points in the paper.

The intention with the paper is primarily to lay stress upon the necessity of making a basin in the first hand hydraulically correct. Else control with other important datas are worthless. If we e.g. have found a certain velocity allowable with regard to scouring we must be sure that this velocity will not be exceeded. In an instable basin a bottom current with twice the mean velocity may be quite normal. Extremely high turbulence can also make the overflow rate value quite misleading. The writer has hence again pointed to the use of dimensionless factors in judging the hydraulic properties of a basin. But he has not suggested certain definite values of the Froude and Reynolds numbers, only given a few examples. Those values must be chosen with regard to the actual, local conditions (type of sediment, size and shape of basin etc.). The influence of different values are furthermore not yet sufficiently known because there is hitherto only a very limited experience available from basins with satisfactory flow conditions. More investigations and research work are requisite.

Next the paper will show that computations with—as we believe—good values of opposing hydraulic factors indicate the advantage of a wide, shallow basin, not a long, narrow one.[2] Also computations by means of the scouring velocity will result in a wide, shallow basin as the most economic one (the least volume has the basin with infinite width).

Finally the paper will show that by means of longitudinal walls (horizontal, vertical or sloping) it is always possible to obtain desired hydraulic values in basins of any shape.

It is true that the writer has tested and suggested basins with long and narrow sections, but the reason is the present engineering status. The writer believes that in the future the trend will be towards wide, shallow basins, plain or arranged in stories. Neither the water distribution, nor the design

1. Civ. Engr., Vattenbyggnadsbyran, Stockholm, Sweden.
2. It should perhaps have been mentioned in the paper, that the long, narrow basin in fig. 8 is one of the answers when using the common linear unit R in the Froude number. The long, narrow basin has 145 times the volume of the wide, shallow basin. If the linear unit is assumed to be the mean depth of the basin, also a wide, shallow basin will be most economic. In the latter case vertical walls would have no influence on the Froude number, but might still be of importance to avoid major disturbances.

of suitable sludge cleaning devices seem to be an obstacle. If the volume of the basin can be reduced to 10% or less by making it wide, a solving of these problems seems worth while.

Prof. CAMP has drawn the conclusion that the paper recommends to establish the maximum velocity by means of an upper limit for Reynolds number, and that this will result in an expensive basin. On the contrary the paper suggests a method to get the desired R at any velocity. It is quite possible to combine the velocity of 12 fpm with an R of 500. But a sediment which allows a velocity of 12 fpm might not be much influenced by a higher R, which then is to be preferred.

Prof. CAMP considers it impossible to distribute the water without using relatively high velocity inlets causing turbulence in the basin. This is not in accordance with the writer's experience, which indicates that good distribution can be obtained with velocities of 1 ft/sec. or less. The writer doubts that an inlet more or less directly from a stirring basin is a better solution than a low loss, multiple "center of gravity" distribution inlet. The problem of distribution is a very important one especially after improving the hydraulic properties of the sedimentation channel.

The effect of an improved low loss inlet and outlet design has been shown in a surprising manner by the experience from the latest built basin (at the town of Vänersborg), where the settled water is now constantly better than by combined ordinary settling and filtration. The residual alumina in the settled effluent (at alum-coagulation without activated silica) from start until now is shown in the table below:-

Date	Temp, °C	Al, ppm	Date	Temp, °C	Al, ppm
Apr. 8	2	0.06	Aug. 3	19.9	0.07
9	2.1	0.06	10	18.9	0.07
12	1.9	0.08	17	19.7	0.10
13	1.9	0.08	24	19.4	0.12
14	2.2	0.07	31	20.6	0.06
16	2.5	0.06	Sept. 8	17.8	0.12
18	2.8	0.06	16	14.2	0.06
19	3.0	0.06	22	13.6	0.06
20	3.3	0.07	29	12.4	0.10
22	2.3	0.06	Oct. 6	11.5	0.06
23	3.0	0.07	12	11.8	0.35
June 9	13.7	-	20	8.3	0.15
16	13.7	0.06	27	6.6	0.08
22	15.2	0.05	Nov. 3	5.0	0.08
29	14.7	0.07	10	5.3	0.06
July 7	15.9	0.08	18	4.3	0.05
15	20.5	0.04	24	3.8	0.03
21	15.8	0.06	Dec. 1	1.9	0.03
27	19.5	0.06	8	2.7	0.09
			5	0.7	0.03
			23	0.2	0.02

The above result has been obtained at a total load 2-3 times as high per horizontal area as at a "conventional" basin with the same water. The rapid sand filters operate about 100 hrs at a rate of 3-4 gals/sq ft. min as compared with about 24 hrs at 2 gals/sq ft. min after the latter basin.

The result opens the possibility of abandoning the rapid sand filter in favor

of rotating filters with micro-screens and hence to make simple, automatic water purification plants.

It seems now safe to conclude that even with a soft, cold humus water a load of 5-8 m/h (3000-5000 gals/sq ft.day) per horizontal unit area is possible for a basin about 15 ft deep with excellent result, which is about the same as with dissolved air flotation under favorable circumstances and much better than with vertical, sludge blanket, sedimentation, which seems not to allow more than perhaps 1000 gals/sq ft.day, and is very sensitive to change in load.

The writer hopes to be able to give operating data from the specially designed sewage basins in the early part of next year.

JOURNAL
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SEWAGE DISPOSAL IN SWEDEN

James Garner[1]
(Proc. Paper 958)

A problem which frequently confronts American sanitary engineers is the rapid absorption and disposal into an existing system of surface water and sewage from a new housing area. The city of Stockholm, which has until recently been falling behind with its sewage and main drainage systems and has more than trebled its population over the last 30 years, has given its engineers a pretty headache.

Traditionally, most European towns resort to artificial drainage only when there is no alternative, especially when drainage and disposal is a cumbersome responsibility no-one much cares about, or has until recently cared about. In Stockholm sewage has always been discharged into the well-known Mälaren and Saltsjön. Indeed, in the first half of the 19th century sewage was simply emptied into the gutters and found its way along with rainwater to the watercourse as best it might. Later a system was introduced under which rainwater was taken to Salstjön by a main and the old gutters were connected with this outfall sometime around 1910.

In true continental style this unhygenic arrangement troubled the Swedes little; indeed the city authorities who had a survey made on their behalf by experts were assured that no harm could result from emptying their sewage in this unpurified manner. Thus the system, with minor modifications and a certain amount of elaboration and extension, continued until 1928, when the watercourses became sufficiently polluted to influence the Swedes to include for the first time some form of purification. Since that date the city's population has grown at a quite remarkable pace. Mainly through the development of suburbs the population has swelled, until it now tops one million, which is more than three times the 1928 figure.

Although the drainage of the original area of the city is based considerably on the old 19th century plan—the open gutters now being, of course, replaced by underground culverts—the suburbs have been dealt with on more modern lines. But the engineers have found it necessary to take into account the natural contours of this rather hilly, broken landscape, in addition to which they have had to connect the new drains with the purification plant at Akeshov.

A feature of the entire planning has been the adoption of combined drainage; that is, both storm water and domestic and industrial wastes are led

Note: Discussion open until September 1, 1956. Paper 958 is part of the copyrighted Journal of the Sanitary Engineering Division of the American Society of Civil Engineers, Vol. 82, No. SA 2, April, 1956.

1. London, England.

away through the same sewer. However, to ease the strain on the system's capacity during heavy storms, some large open areas on the outskirts of the city have been provided with separate drainage which siphons off surplus water.

One of the outlying areas of the city, which is now being developed into a residential quarter of some 60,000 inhabitants, is being tunneled. The tunnels, which are about 50 square feet in cross section, not only form the main drainage for this suburb, but also themselves act as a basin to hold surplus water. Thus it has been found unnecessary to expand the lower part of the system to accommodate any storms or to modify the existing tunnel with which it is connected. This latter tunnel, which was built in the 1930s, leads to the Akeshov plant.

The terrain under which the new system has been driven is irregular, consequently in places the rock level has fallen so low that the covering has been inadequate and it has been necessary to use soft earth techniques. The cut and cover method has been mainly adopted where the rock level proved too low, but in certain districts it was preferred to use pipe driving. This is more expensive than trenching but where there were obstacles on the surface it was necessary. Pipes were forced through the clay, which was fairly soft and without stones, and the soil was then extracted.

At one point of the system, where the new drains connected up with the old, it was found that the rock level was too low for tunneling, whilst underground water and the softness of the clay made deep trenching difficult. Therefore it was decided to drive pipes through and by this means 150 feet of culvert with a diameter of seven feet, was commenced.

The tunnels themselves, however, which form the main part of the work are nearly finished. Blasting operations continue from two directions simultaneously, and the estimated cost of tunneling has been about $45 per foot. Since the present schedule of work was commenced eight years ago nearly two million cubic feet of rock has been excavated, this residue being used for highway construction purposes.

Labour in Sweden, as in Britain, Western Germany, and many European countries, is scarce; therefore the Swedes have used many mechanical aids over and above those they would normally have used. A light railway removes the excavated rock, and this is operated by compression-ignition engine necessitating a constantly pumped supply of fresh air as an important safeguard against CO fumes. At the surface the residue is carted off by mechanically loaded trucks to the crusher.

Smaller branch tunnels and bores, which connect with the local drainage lines, have been integrated with the tunnel system; expansion chambers have also been incorporated.

An extra burden upon the shoulders of the planners (an overloved word in the vocabulary of Europe's economists, many of whom have left wing sympathies!) has been imposed by the decision to earmark further adjoining tracts of land for future exploitation and development. Thus the drainage system has to be able to accept a likely increase on the present volume of waste in the next decade or so.

Economic circumstances have compelled further departures from tradition. The Swedes have decided that the cost-cutting multi-drainage type of installation, as opposed to the older system under which each house had its own private drains, is preferable. Further, the broad roads now being built, together with the wide verges, add considerably to the mileage of service

piping, making multi-drainage all the more attractive a proposition. The Swedes are communal minded in other ways too; many district hot water and heating schemes are afoot.

This development and future expansion around Stockholm indicates that the Old World still has progressive tendencies. Sweden may yet show us an unparalleled cosmopolis of Scandinavia.

DIVISION ACTIVITIES

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

PUBLICATION NEWS

Ross E. McKinney
Chairman, Publications Committee

By this time everyone is aware of the change in Proceedings. Instead of being issued as separates, Proceedings are being issued as a regular bimonthly bound journal. While the format of the separates is being retained for those who wish to separate and keep only certain articles, binding of each issue will be an aid to those desiring to keep all of the articles together in a manageable form. It is hoped that this system will satisfy all the members of the Sanitary Engineering Division of ASCE.

With a regular publication schedule your Publication Committee will be faced with the task of securing and reviewing a sufficient number of papers to meet that schedule. The number of sanitary engineering papers presented at the various ASCE meetings is not enough to meet a bimonthly publication schedule. Therefore, if the Sanitary Engineering Division is to have a good technical journal, the members of the SED must submit enough papers to meet the demand. For many members of the SED the Proceedings are the major return for the dues paid to the ASCE each year. In order to obtain the maximum return for the dues, each member should take it upon himself to determine if he could not make a contribution to the Proceedings. It is impossible for your Publication Committee to contact each one of you in person to see if you have something of value to write up. Therefore, your Publication Committee would like you to consider this a personal invitation for submitting a paper for the Proceedings.

The question has arisen as to what type of papers should be published by the SED in their Proceedings. The Publication Committee will consider any paper dealing with the technical aspects of the over-all field of Sanitary Engineering. The field of Sanitary Engineering is extremely broad and the publications should reflect this breadth. All papers will receive equal consideration regardless of their topic as long as the paper is of technical interest to Sanitary Engineers.

One of the major problems with papers submitted for publication is their failure to follow the form and the mechanical details set forth in Separate No. 290. The ASCE does not maintain an editorial staff to rework papers which do not conform to the specifications in Separate 290. Each division is required to have a Publication Committee to review the papers submitted and to make sure that all of the requirements have been satisfied. Failure

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to properly prepare the paper initially results in considerable delay in publication. A summary of the requirements in Separate 290 are given below.

1. Three copies of the manuscript, the original and two copies, shall be submitted to the Manager, Technical Publications, A.S.C.E., 33 W. 39th St., New York 18, N. Y.
2. The manuscript should be typed, double spaced, on one side of 8½ x 11 paper.
3. The title of the paper should not exceed a total of 50 characters and spaces.
4. The author's name and membership grade should appear below the title with a footnote as to present position.
5. A 50-word summary of the paper should precede the body of the paper.
6. A clear, concise statement or paragraph should introduce the problem to the reader.
7. The body of the paper should include the methods employed in the study, the theoretical and practical concepts, the data, and a discussion of the data and the problem.
8. A clear statement of the conclusions to be drawn from the paper should end the paper.
9. No line of mathematics can exceed 6½ inches and all equations must be drawn in India ink with capital letters at least 3/16 inch high.
10. All tables should have a one inch margin along each side and a 1/2 inch margin at the top and the bottom. Each table shall be typed on a single 8½ x 11 inch sheet.
11. All figures must be drawn in black ink on 8½ x 11 inch sheets with the same margin as for tables. The original figures must be submitted. The title of the figure and the figure numbers must be included as part of the figure. The lettering and the details of the figure should be large enough that it can be reduced 69 per cent and still be readable.
12. Failure to conform to any of these specifications will lead to rejection of the paper or delay until such time as the paper has been changed to conform to the required form.

The success of the SED Proceedings lies with the interests of the members. The Publication Committee cannot obtain sufficient papers to meet a bi-monthly schedule without your help and support. It is up to you to make the new Proceedings a success.

DID YOU KNOW THAT—

Dr. D. B. Smith, on leave from the University of Florida, has been appointed Director of the Florida Water Resources Study Commission and has established headquarters for the Commission on the Campus.

The Water Resources Study Commission was created by the State legislature to conduct a study to determine whether or not there is a need for a comprehensive water law in Florida administered by a board, and if so, the extent of jurisdiction of the board and details of operation such as relationships with other agencies.

The ninth Municipal & Sanitary Engineering Conference will be held on the Campus of the University of Florida April 17-18. Air Pollution will be the theme of the Conference.

The Harvard Business Review (July - August 1955) carried a very interesting article entitled "Stimulate Your Executives to Think Creatively," by Frederic D. Randall. All Executives, and those who hope to be Executives, will find the explanation of what makes people think creatively (and what factors interfere with creative thinking) very interesting.

Professor Robert E. Stiemke, Director of the Civil Engineering School at Georgia Tech., has recently assumed his duties as President of the Georgia Engineering Society.

Dr. Marvin Granstrom of the University of North Carolina is at present working in Peru. His assignment is part of the University's cooperative program in sanitary engineering education.

Mark D. Hollis, Assistant Surgeon General and Chief Engineer of the U.S. Public Health Service, was awarded the honorary Doctor of Science degree by the University of Florida at the mid-year commencement exercises. Mr. Hollis also was principal commencement speaker, addressing the 500 graduates on the subject, "Advancing Technology - A Dynamic Impact on Environment and Health."

In his address Mr. Hollis stressed the increasing rate of new technological developments, the associated build-up throughout the country of metropolitan concentrations of population and industry, and the deterioration of community air and water resources resulting from heavy discharges of wastes. Projection of trends indicates that the problem of controlling waste discharges, to limit discharges to amounts which will not overtax the natural purification capacities of air and water resources, will become increasingly difficult and will be a limiting factor in future metropolitan development.

R. G. Kincaid of Burns and McDonnell, Kansas City Consulting Engineers, is making a preliminary survey for the development of a water supply for Athens, Greece.

The Sanitary Engineering Division was well represented in the recent election of L. W. Bremser and W. G. Riddle to the offices of 2nd Vice-President and Secretary-Treasurer, respectively, of the Kansas City Section of the ASCE. Sanitary Engineering Division members appointed as Committee Chairman include R. O. Davis, Program; F. C. Wilson, Membership; R. F. Banker, Attendance.

Henry B. Steeg, Prominent Consulting Engineer of Indianapolis, Indiana, passed away suddenly on December 17, 1955. Mr. Steeg was head of the firm Henry B. Steeg and Associates and was quite active in ASCE affairs.

Dr. Gerald H. Teletzke, formerly an instructor of Civil Engineering at the University of Wisconsin, has been appointed Assistant Professor of Sanitary Engineering at Purdue University. Dr. Teletzke will participate in the teaching program and assist Professor D. E. Bloodgood in the development of a broad program of basic and applied research especially in the field of industrial and atomic-waste disposal.

R. G. Paulette, Ass't. Manager of the Industrial Service Division of Municipal Service Company in Kansas City, presented a discussion before the Kansas City Section in February entitled "Industry's approach to their waste treatment problem." The need for recognizing the existence of a problem (preferably prior to citation by a pollution control authority), the types of surveys to be conducted by specialists in the field of waste treatment, and the need for proper study of the data collected during the study, were stressed as points to be considered in the preliminary report. Methods of reducing the waste problem were also discussed.

An article is being prepared by Mr. Paulette for later publication in this journal.

The Toledo, Ohio, section was addressed by Edmund B. Besselievre on February 1 on the topic "Going Abroad for Business." He reviewed his work, including many sanitary engineering projects in Latin America, South Africa, Europe, and the Far East.

Fred H. Waring, Chief Sanitary Engineer, Ohio State Department of Health, addressed the Ohio Section at its November 17, 1955, meeting. Mr. Waring discussed Ohio's water pollution control program with particular reference to accomplishment under the new State law which went into effect in 1952. He drew a parallel with the water pollution control program which was in effect when he joined the Department in 1916. He stated that the principal incentive for water pollution control at that time was protection of public water supply and prevention of lawsuits by downstream riparian owners.

John C. Bumstead, former Secretary of the SED and co-editor of the SED News-letter, has recently taken a position with the City of Cincinnati as assistant to A. D. Caster, Sewage Disposal Engineer, and present Secretary of the SED. It will be remembered that this relationship between John and Art was the other way around when John was Secretary of the SED and Art worked for him as Assistant Secretary.

Three members have been added to the Publications Committee recently. They are William A. Cowley, William Dobbins, and William Ingram. Why don't you get into the swim by offering to serve on a Committee?

On March 1, 2, and 3, the Committee on Preparation of a Manual of Practice on Sanitary Storm Sewer Design and Construction held its final meeting for completion of a text of the Manual. We should all look forward to the future publication of this Manual.

Many Sanitary Engineers attended and took an active part in the program of the very successful Hydraulic Conference sponsored by the Kansas City Section late in November.

John A. Strang and Edmund Wilkes, Jr., were presented at the February 14 meeting of the Kansas City Section as candidates for life membership in the Society. The presentation to Mr. Strang, Midwest Area Manager of Wallace & Tiernan, Inc., and Mr. Wilkes, of Black & Veatch Consulting Engineers, was made by the President of the Section, Louis G. Feil.

Ray E. Lawrence, of Black & Veatch and SED Executive Committee Member, was a member of the group headed by Kansas City Mayor H. Roe Bartle which made a 24-day good-will tour of South American countries.

L. E. Ordelhiede, former Chief Engineer of the Missouri Division of Health, resigned his position as Executive Director of the Metropolitan St. Louis Sewer District, and is now associated with the Consulting Firm of Haskins, Riddle, and Sharp in Kansas City. Other Engineer-additions to this firm include Marlowe Mankford, former City Engineer at Hannibal and Springfield, Mo., and Ulysses, Kansas, John A. Zaffel, formerly with the Corps of Engineers and recently in Phoenix, Arizona, and Dan Weiner, a former associate of this firm who has returned after 2 years in Saigon, Indo-China, with the U. S. Public Health Service.

Public Health Service engineers from the San Francisco Regional Office of the U. S. Department of Health, Education, and Welfare assisted State and local personnel in inspection of damaged water supply and sewage disposal systems following the worst flood in decades in northern California, Nevada, Oregon, and Washington.

Six sanitary engineering personnel trained in the field of housing rehabilitation were also temporarily assigned to the American National Red Cross to assist with estimates of the cost of rehabilitation of the approximately 50,000 damaged homes in the flood-affected areas.

Leo F. Krapp has been appointed by the City of Cincinnati to the position of Superintendent of Maintenance in the Sewage Disposal Section. Leo will be responsible for the maintenance of buildings, equipment, and grounds for Cincinnati's three disposal plants and appurtenant structures.

Vincent B. Lamourex, Public Health Service Officer, who has been Assistant Chief, Near East and Africa Branch, Public Health Division, International Cooporation Administration, at Washington, D. C., departed on February 1 to become a member of the U. S. Operations Mission to Uruguay, where he will advise on urban and rural sanitary engineering problems. He also will instruct university students in public-health engineering.

Mr. Lamoureux's new address will be: Health Welfare and Housing Field Party, USOM of Uruguay, c/o American Embassy, Montevideo, Uruguay.

At the time this Section of the Journal was prepared, the Committee on Public Works of the House of Representatives had announced that hearings on the Federal Water Pollution Control Bill would be held March 12-14, 1956. The bill, known as S. 890, was introduced and passed by the Senate during the 1st Session but was not voted upon by the House.

The Resolution in support of the Act passed by the SED Executive Committee has been approved by the ASCE Board of Director's and forwarded to Washington.

Joseph H. Ehlers, Assistant Commissioner for Technical Services, Urban Renewal Administration, at Washington, D. C., has been commissioned a Sanitary Engineer Director in the Public Health Service Commissioned Reserve.

Immediately prior to taking his position with the Urban Renewal Administration, Mr. Ehlers was field representative of the American Society of Civil Engineers at Washington, D. C. He is presently co-secretary of the National Joint Committee of the American Society of Civil Engineers and the American Institute of Architects, and Chairman of the Advisory Committee on Engineers of the U. S. Civil Service Commission.

Since the last issue Samuel W. Shafer & John S. Bethel, Jr. have agreed to serve as Assistant Editors of the Division Affairs Section of the Journal.

Mr. Shafer is Assistant Superintendent of the City of Cincinnati, Ohio's Mill Creek Sewage Works and Mr. Bethel is a member of the Consulting Engineering Firm of Metcalf & Eddy in Boston, Massachusetts. How about the Southwest and the Northwest? Any one willing to spend a little time to collect news items and other contributions and report them bi-monthly? The success of this section depends on widespread contributions from all members.

A National Conference on Water Resources Policy was held in St. Louis, Missouri on January 24-25, 1956. The Conference sponsored jointly by the Chamber of Commerce of the United States, National Water Conservation Conference and the Engineers Joint Council indicated that there is a trend toward general agreement on many elements of National Water Policy. A few features, however, are still highly controversial.

Early in their careers, many sanitary engineers find that they must deal effectively with people. They must meet the public and convince them of the need for new installations or programs; they must gain the cooperation of people in other public health professions; or their jobs may place them in the position of having to administer extensive programs involving the activities of numerous people. Unfortunately, the professional education of most sanitary engineers does not prepare them for these functions. The individual must rely upon his native ability to see him through. It is hoped that the following article will be of interest to all members of the profession. -- E. R. Hendrickson

TENSIONS AND FRUSTRATIONS IN ADMINISTRATION

(Abstracted From an Article by
Dr. A. W. Combs, University of Florida.)

Unquestionably, administration is a difficult job. When an administrator feels stresses and strains, almost without exception he studies his job in search of answers to his problem. Recent research on this problem, however, has led to a quite different conclusion; that the tensions and frustrations of administration lie not in the job but in the administrator himself. Tensions rarely exist with respect to things. Physical objects can be counted on to behave or respond according to a fixed pattern. Most of the tensions and frustrations of administration arise from within the administrators themselves or from their relationship with other people. The sources seem to lie in three major areas: the administrator himself, the administrator's concept of other people and the interrelationships of these two.

CAUSES OF FRUSTRATION WITHIN THE INDIVIDUAL

Lack of Consistent Basic Philosophy

This seems to be one of the major causes of strains and tensions in administration. A philosophy gives consistency to behavior. Without some kind of consistent basic philosophy, one's behavior is likely to be inconsistent and erratic. It should be pointed out that there is a great deal of difference between behaving in terms of a personal philosophy and knowing about philosophy. It is quite possible to know about many philosophies but to have no philosophy of one's own. Indeed, it is even possible that knowing about many philosophies may leave one in so much doubt about what he believes that his behavior is highly erratic.

The lack of a philosophy shows itself in the behavior of the administrator in inconsistencies. Without a firm basic philosophy, administrators may be heard saying "personnel should be allowed to.... but...," or "this is a staff problem but....."

An inconsistent philosophy not only leads to inconsistent behavior but results in important adverse effects on other people. Others never know where the administrator stands and are never quite sure what he is going to do. When authority wavers, people in lower echelons have little to base

their decisions upon. Consequently, their behavior in turn becomes hesitant and uncertain. This is likely to give the administrator the feeling that people are constantly letting him down and communication between the administrator and others may become extremely difficult. People can adjust to almost anything if it stands still.

Without a firm basic philosophy, the administrator is left without a guide to determine what is important. As a result, everything has to be decided every day. This can lead to a vicious circle of circumstances which will preclude the development of a philosophy.

Lack of a Clearly Understood System of Values

Many of us claim one set of values while behaving as though we held another. There is considerable difference between knowing what one should value and truly holding these values for himself. When our values are unclear, it is difficult for us to decide what is important. As a result, every pressure to which we are subjected must be handled at once. The administrator who has not decided for himself the important values in his work is at the mercy of his job. He must handle very situation as it arises. Because he has handled everything, his colleagues or employees expect him to do so. Those who surround him assume that this is the way he feels his job should be done. If this continues, the administrator may find himself so busy he does not have time to think about what is important!

In a similar fashion, the administrator without a clear sense of values finds himself at the mercy of the circumstances in which he operates. Administration becomes a matter of opportunism rather than direction. This may result in a vicious circle of continuously increasing pressure as the administrator finds it necessary to handle all details.

Lack of a consistent value system in the administrator often results in behavior puzzling to his subordinates. Inconsistent behavior creates confusion in the minds of other people and may result in a lack of confidence, fear and resentment toward the administrator. When we know what people value, it is not difficult to predict how they will behave in a given situation.

CONCEPTS OF OTHER PEOPLE

The Fetish of Objectivity

The worship of objectivity seems almost to be a disease of administrators. Whenever it is necessary to deal with other people, the cry goes up "get the facts." This is based on the very erroneous assumption that people behave according to the facts. Actually, people never behave according to the facts as we see them. A fact is not what is, it is what you think is so. For example, if a worker thinks her foreman is unfair, it does not matter very much insofar as her behavior is concerned whether he is or not. So far as the worker is concerned, what she thinks is so is so. If we fail to understand this important concept, we may fail entirely in being able to deal with people.

People behave according to how things seem to them, not according to how things seem to administrators. Failure of the administrator to understand this may lead to confusion and frustration in his dealings with the people about him. It is necessary to recognize that the administrator and his subordinates live in different worlds and do not see things alike. The

administrator who takes time to prepare his people for changes and takes the time to tell them the whys and wherefores can eliminate many frustrating experiences.

Behavior is a Function of Values and Needs

Part of the administrator's job is to visualize things that need to be done. An attempt to incorporate such changes, however, in the absence of need as seen by others may result in frustration. For example, a need for a new school building in a community will only be attempted when the community sees the need. If an administrator visualizes the need and the community does not, it will only result in further frustration for the administrator.

Many of the stresses and strains of administration occur because of a failure to communicate in such a way that need seen by one is similarly seen by others. In many instances, the individual on the receiving end feels that the administrator is being picayunish or obstinate, and such circumstances will result in a loss of confidence in the administrator. When people do not understand, they may feel themselves victims of coercion and seemingly unreasonable orders.

The Stockyard Approach to Human Relationships

There exist today two general approaches to the problems of dealing with other people and with the question of leadership. Each depends upon what we think other people are like.

The first might be described as the stockyard approach. It consists of going ahead of the herd and closing all the gates which the cattle are not to use and opening those in the direction you wish to have them move. When this has been accomplished, one gets behind the herd and creates some kind of annoyance to put them in motion. It is a method of dealing with people based upon fencing them in. This technique is frequently used in our society and, while it works very well with cattle, it often breaks down with human beings.

Human beings, being smarter than cattle, are just as likely to jump the fences, open gates that we never knew existed, or generally refuse to go where we wish. To use this technique effectively, moreover, requires that somebody know the "right" path that other people should follow. This leads directly to a "great man" philosophy. The person who uses this technique must know the right answers. He cannot fail. He is responsible for everything. Responsibility cannot be shared. Unfortunately, such an approach has a negative effect upon the people who are led for it creates an attitude of dependence and a lack of responsible action. It requires of the administrator, furthermore, that he make constant decisions. Under such pressure, failure is intolerable and when it occurs produces tremendous feelings of guilt.

Such a force and coercion method of dealing with other people has its effects upon subordinates as well. People do not like to be forced and coerced even when it is done with a kindly hand. As a result, they are likely to feel resentment, anger, hostility, and at the first opportunity break loose from such coercion. This causes the administrator to feel a loss of prestige. In desperation, he may then be led to make the mistake of increasing the force and coercion which produces uncooperative behavior on the part of subordinates and another vicious circle may be started.

The Facilitating Approach

This approach is quite different from the stockyard approach we have just been discussing. It begins with the assumption that a group is seeking an answer together. The leader's job is one of assisting or facilitating the process. It begins with the attitude: "Let us go together, through conference and discussion." In this kind of an approach responsibility is shared, communication is facilitated, need is established, and occasional failure is expected. Although it may be somewhat slower than the stockyard approach to a problem, in the long run it is likely to be more effective. Even more important for the administrator, it is likely to produce, stimulate, and encourage new ideas rather than force the defense of ones already in existence. Better answers are likely to be produced because there are more heads involved. The use of this approach does not require that the administrator give up his own personality and thinking; but neither does he over-value it or create his own frustrations.

Failure of Communication

Understanding among people can only occur on the basis of common meanings. We can only understand each other when what has a meaning to you has a similar meaning to me. Failure to achieve this kind of common thinking with people who surround us may result in frustration for ourselves. The administrator who is unable to communicate with other people finds himself forced to protect himself, and this is a further deterrent to effective communication. Administrators frequently complain of the lineliness of their jobs, but a great deal of this loneliness may be of their own making.

It is important for us to recognize that the discovery of personal meaning takes time. When people do not perceive clearly, they cannot behave clearly or precisely. Haste in our efforts at communication may produce half perceptions on the part of the people with whom we deal and result in only half behavior. Unfortunately, such reactions many times cause the administrator to apply more pressure which in turn results in greater resistance and greater frustrations. Clear and precise actions on the part of the administrator may be the best defense he can muster.

Appeasement, Acceptance and Attack

There is a common feeling that there are only two ways in which we can deal with other people—appeasement or attack. In our society, appeasement is considered shameful and may result in a loss of prestige. Attack saves face but may create confusion, fear and distrust on the part of the attacked. Many of the tensions and suspicions of administrators appear due to this dichotomous approach to the problem of dealing with others.

Many have apparently overlooked the third possibility of acceptance. This is a position of honesty and integrity, not to be confused with resignation. It is a position which states: "I will not attack you but neither will I permit you to push me around." One who assumes this attitude is not placed in a position which requires either attack or appeasement. The individual who behaves in this way creates respect in others and reduces his own frustrations to a minimum.

Policy and Action

A frequent source of stress and strain in administrators seems to be the failure to recognize that policy is always the prerogative of the group. Policies in a democratic society are made by the people. Executive action is the function of the administrator. Many of the frustrations of administrators appear to be due to a breakdown in the mind of the administrator of this important distinction. By the actions he takes, the administrator may insidiously encroach upon the policy-making function of his group. This has the effect of disfranchising the group and leaving people unable to judge what is expected of them or how they should behave. Under these circumstances, people have little choice but to become completely dependent upon the administrator, while at the same time, becoming highly suspicious of his bad faith and inconsistency.

Faith in Others

The effective administrator needs to have the faith of other people in him. Other people, however, are likely to have little faith in one unless the feeling is reciprocal. It is important to recognize that our behavior affects other people and if we make others feel liked, wanted, accepted and able, they are very likely to contribute to our feelings of this sort.

All behavior exists within certain limitations. Many of the frustrations, stresses and strains of administration result from the failure of adminis trators themselves to clearly establish the limits of the situations within which they and their subordinates are expected to operate.

PRESIDENTIAL ADVISORY COMMITTEE ON WATER RESOURCES POLICY MAKES REPORT

All Sanitary Engineers concerned with the field of Water Resources will be interested in the Report made to the President of the United States by his Advisory Committee on Water Resources Policy. In the letter of transmittal of this report to Congress on January 17, 1956, the President stated that "The policies set out in the report embody a framework within which the Federal Government, with State and Local Governments and other non-federal interests, may cooperate to develop our water resources."

The document, which is brief (less than 50 pages) and concise, outlines the problem in general and considers the following specific problems:

- Collection and evaluation of basic data
- Planning for use and development of water resources
- Board of review for water resources
- Federal, State, and local relationships with reference to the use and control of water
- Priority of use of water
- Evaluation of water resources projects
- Authorization of projects
- Participation in costs

In transmitting the report to the President, the Committee recognized that

many of the issues were highly controversial and had been the subject of public debate for many years. The Committee set forth its proposals but recognizes that these proposals "will not immediately be acceptable to all persons and organizations concerned with water resources." The Committee stated that it believed, however, that the recommendations made in the report were fair and equitable and in the best interests of the Nation.

The report, which in itself is quite brief, cannot be summarized without the risk of misinterpretation. The entire report is recommended reading for Engineers with interests in the field of water resources. The report has appeared in several forms, but probably the most available is House Document 315 (84th Congress, 2nd Session) which can be obtained from your Congressman by simply writing and asking for it.

Letter to the Editor

EDITOR'S NOTE: We have invited participation in this section of the journal and we do not want to discourage our contributors by too much editorial censorship. The writer of this letter, who's name is withheld at his request, has obviously written this letter with tongue in cheek. It is presented here since it illustrates the fact that the Sanitary Engineer is always facing new problems - and in a way that may stimulate a smile or two. PWR

Dear Editor:

Lately I have been hearing and reading quite a bit about space stations, rocket ships and inter-planetary travel. One little item has been plaguing me, and so far I have not been able to visualize the answer. And for that matter nothing on the problem has been mentioned anywhere.

I know that the science of waste removal and disposal has advanced at a very rapid rate - but are we able to cope with the problem of bodily discharges at conditions of minimal gravitational force. We have been told that the shoes of the space travelers will be magnetized to keep him from floating around the space ship - but that is about all we can magnetize. It appears difficult enough to provide food for the space voyager - but on any extended trip, as all space travel must be, what will be done with the human waste products?

Somehow my aesthetic concepts of the heavens are being destroyed by the obvious thought that we can discharge the wastes to the outside of the hull where they will forever float around in limbo.

I wonder if we should ask science fiction writers to offer suggestions, or maybe some of our members might have some ideas on the subject.

For cleaner space travel, I remain

Yours truly,

Please cooperate by sending news items of interest to:

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

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c. Discussion of several papers, grouped by Divisions.

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